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THE PREDICTION OF THE FREEZE/THAW DURABILITY OF COARSE AGGREGATE IN CONCRETE BY MERCURY INTRUSION POROSIMETRY

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Interim Report

THE PREDICTION OF THE FREEZE/THAW DURABILITY OF COARSE AGGREGATE IN CONCRETE BY MERCURY INTRUSION POROSIMETRY

TO: H. L. Michael, Director
Joint Highway Research Project

November 25, 1980

Project: C-36-47N

FROM: D. N. Winslow, Research Associate
Joint Highway Research Project

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The attached Interim Report titled "The Prediction of the Freeze/Thaw Durability of Coarse Aggregate in Concrete by Mercury Intrusion Porosimetry" is the first on Phase II of the HPR Part II Study titled "Improvement of Non-Durable Aggregates in Portland Cement Concrete". The Report has been prepared by Ms. Meredith Nickels Lindgren, Graduate Instructor in Research on our staff under the direction of Professor D. N. Winslow.

The Report reports verification and refinement of the validity of the findings of Kaneuji in Phase I of this research of a correlation between the pore size distribution of coarse aggregate and durability. The findings will be of value to ISHC in the development of improved specifications for coarse aggregate so as to insure long-term durability of concrete.

The Report after review is planned for publication.

Respectfully submitted,

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by

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16. Abstract

D-cracking is a serious problem of concrete pavements in freezing climates. The main cause of this distress is the coarse aggregate. Kaneuji (1) determined a correlation between the pore size distribution (from mercury intrusion) of an aggregate and the freeze-thaw durability of concrete using the same aggregate. He developed an Expected Durability Factor EDF, used to determine whether an aggregate can be expected to be durable or nondurable. The present study was designed to refine the validity of Kaneuji's correlation and better define the pore structure criteria by which to predict the performance of an aggregate.

Aggregates from fifty-two Indiana highway cores were tested, as were five rock samples supplied by the Portland Cement Association. The EDF values were determined from the pore size distributions, and an "average" value was assigned to each pavement associated with the cores. These values were then compared with the field performance of the pavement to ascertain the borderline between EDF values for durable for nondurable aggregates. A good correlation between the field performance and the "average" EDF values was found. A pavement will be durable if its coarse aggregate has an EDF value greater than 50 for 90% or more of the aggregate. This criterion applies to stone and gravel aggregates with a maximum size of 1-1/2" to 2-1/2". The pavement will be durable for at least thirty years.

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HIGHLIGHT SUMMARY

D-cracking is a serious problem of concrete pavements in freezing climates. The main cause of this distress is the coarse aggregate. Kaneuji (1) determined a correlation between the pore size distribution (from mercury intrusion) of an aggregate and the freeze/thaw durability of concrete using the same aggregate. He developed an Expected Durability Factor EDF, used to determine whether an aggregate can be expected to be durable or nondurable. The present study was designed to refine the validity of Kaneuji's correlation and better define the pore structure criteria by which to predict the performance of an aggregate.

Aggregates from fifty-two Indiana highway cores were tested, as were five rock samples supplied by the Portland Cement Association. The EDF values were determined from the pore size distributions, and an "average" value was assigned to each pavement associated with the cores. These values were then compared with the field performance of the pavement to ascertain the borderline between EDF values for durable and for nondurable aggregates. A good correlation between the field performance and the "average" EDF values was found. A pavement will be durable if its coarse aggregate has an EDF value greater than

50 for 90% or more of the aggregate. This criterion applies to stone and gravel aggregates with a maximum size of 1½" to 2½". The pavement will be durable for at least thirty years.

An estimated absorption was calculated for each aggregate tested with mercury intrusion, and an average estimated absorption was computed for each core. These values were compared with the Indiana State Highway Specifications. The EDF value was found to be a good indicator of how an aggregate will perform.

INTRODUCTION AND LITERATURE REVIEW

Review of Previous Work

Deterioration Line Cracking, or D-cracking, is a problem of concrete highways in freezing climates. This phenomenon is usually characterized by a series of cracks parallel and adjacent to joints and edges of portland cement concrete pavements. It is caused by cyclic freezing and thawing. Water enters the concrete, frequently at the joints and edges, and saturates the aggregate. Freezing causes expansion of the water and creates stresses in the aggregate and matrix. If these stresses exceed the tensile strength of the material, cracking will result. This distress sometimes begins at the bottom of the slab and progresses upward until the entire slab is affected.

Kaneuji (1) found a correlation between the frost durability of concrete laboratory specimens and the pore size distribution of their coarse aggregate, as measured by mercury intrusion. He analyzed the freeze-thaw data, obtained per ASTM C666A, and the mercury intrusion results and developed a correlation equation, as follows:

$$EDF = K_1/PV + K_2(MD) + K_3 \tag{1}$$

where: EDF = Expected Durability Factor

PV = intruded pore volume of pores larger than 45 A^o (4.5 nm) in diameter, expressed in cc/g

MD = median diameter of pores larger than 45 A^o in diameter, expressed in μm

K_1, K_2, and K_3 are constants with values of 0.579, 6.24, and 3.04 respectively

Pore volume and median pore diameter are important aggregate properties affecting frost durability. The larger the pore volume, the lower the durability will be; the smaller the median pore diameter, the lower the durability will be. However, pores with diameters smaller than 45 A^{o} do not affect the durability; owing to their small size, water presumably doesn't freeze in them at ordinary temperatures.

The correlation equation given by equation (1) is shown in Figure 1. Aggregates with a median diameter, MD, smaller than 0.1 micron have EDF values that are largely controlled by the pore volume, PV. As the MD values increase EDF values gradually increase.

Based on a few comparisons with actual field performance, Kaneuji set the following tentative borderlines for classifying coarse aggregate:

EDF	Predicted Durability
0-40	Nondurable
40-50	Marginal
> 50	Durable

Aggregates with a wide range of pore volumes and pore sizes were tested. Most of the natural aggregate pore size distributions for Indiana aggregate were covered. The amount and distribution of pores larger than about 15 microns varied, but if they are well distributed throughout the aggregate and are not saturated with water, they may work similarly to air voids in cement paste. How these pores improve frost durability is uncertain; Kaneuji therefore recommended that the EDF calculation be based on pore size distributions in the range smaller than 15 microns and that larger pores be counted as uncertainty that

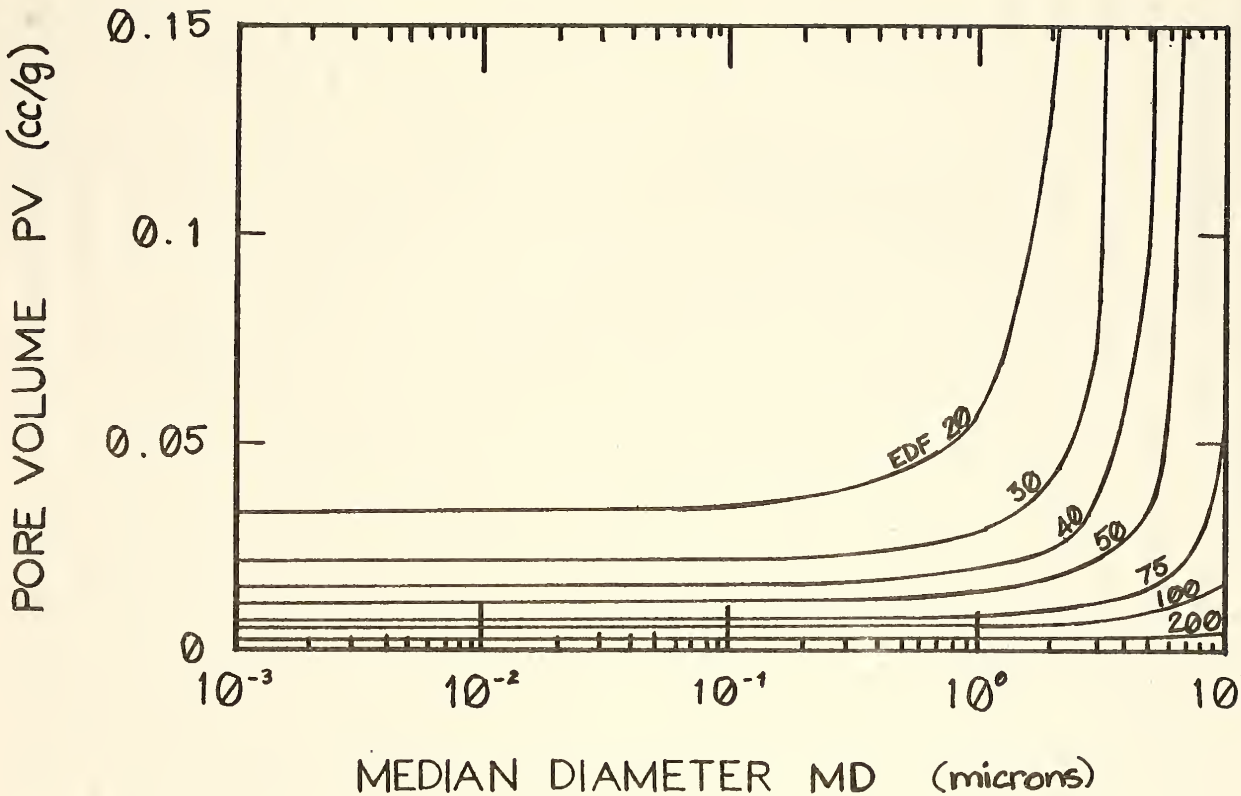

Figure 1. Correlation Equation

probably results in a more durable aggregate.

For pore volumes smaller than 0.012 cc/g, the aggregate will have an EDF value greater than fifty no matter how small the median pore size. If the MD is greater than 6.80 microns, the aggregate's EDF value will be greater than fifty, even if the pore volume is greater than 0.10 cc/g. Kaneuji also determined that if the proportion of aggregate components with low EDF values is small, the concrete seems to have better durability as a whole. This idea is especially significant in the case of gravel samples. Owing to their great heterogeneity, a wide range of EDF values can be expected for a gravel.

In the original analysis, a spread factor, SF, was included in the correlation equation. The SF is the ratio of the 25-percentile pore diameter to the median diameter. It is associated with the shape of the pore size distribution curve, or the range of pore sizes that are present. The range of SF values for Kaneuji's samples was not wide, so the SF was not included in the final equation. It may be applicable to very porous aggregates or to slags, as will be discussed later.

Other studies have been done on the problem of D-cracking and how to remedy the situation. One such study was recently done by the Illinois Department of Transportation (2). They surveyed over 3,000 miles of concrete pavement and evaluated the aggregate using the Pore Index Test, a method for intruding water into the pores of the aggregate to determine the frost durability of the aggregate (2,3). This test measures the volume, in milliliters, of water injected into the pores of a 9000 gram sample of coarse aggregate under a constant pressure of 35 psi. The maximum size of the aggregate is 3/4 inch. One minute after first

applying the constant pressure, the change in volume of the water is recorded. This volume change is called the primary load and is not used in the pore index test results. The purpose of the primary load is to fill the macro-pores of the aggregate, which are believed to be beneficial. A well-developed macro-pore system probably behaves in much the same manner as air voids in cement paste.

After fifteen minutes have elapsed another water intrusion reading is taken. The volume of water injected between one and fifteen minutes is called the secondary load and it supposedly represents the amount of water intruded into the micro-pores of the aggregate. This secondary load is called the Pore Index. A secondary load of about 27 ml or more is supposed to be indicative of negative aggregate quality and may correlate with lack of frost durability.

The Illinois DOT study compared the pore index test results to field performance and to freeze-thaw test results. The field performance was characterized by assigning a number from 0 to 3 to the pavement in question. A rating of 0 meant that no D-cracking was observed and a rating of 3 indicated severe D-cracking. A plot of the D-cracking rating versus the years of service was made, as shown in Figure 2. Field performance zones were determined based on data obtained through the study. Zone A indicated good field performance, Zone B was a marginal range, and Zone C indicated poor performance. A rating of A, B, or C was assigned to an aggregate source, depending on the zone in which most of the data points for the pavement were located.

After determining the zones of field performance, the pore index test results were compared with the field performance. The following correlation was indicated:

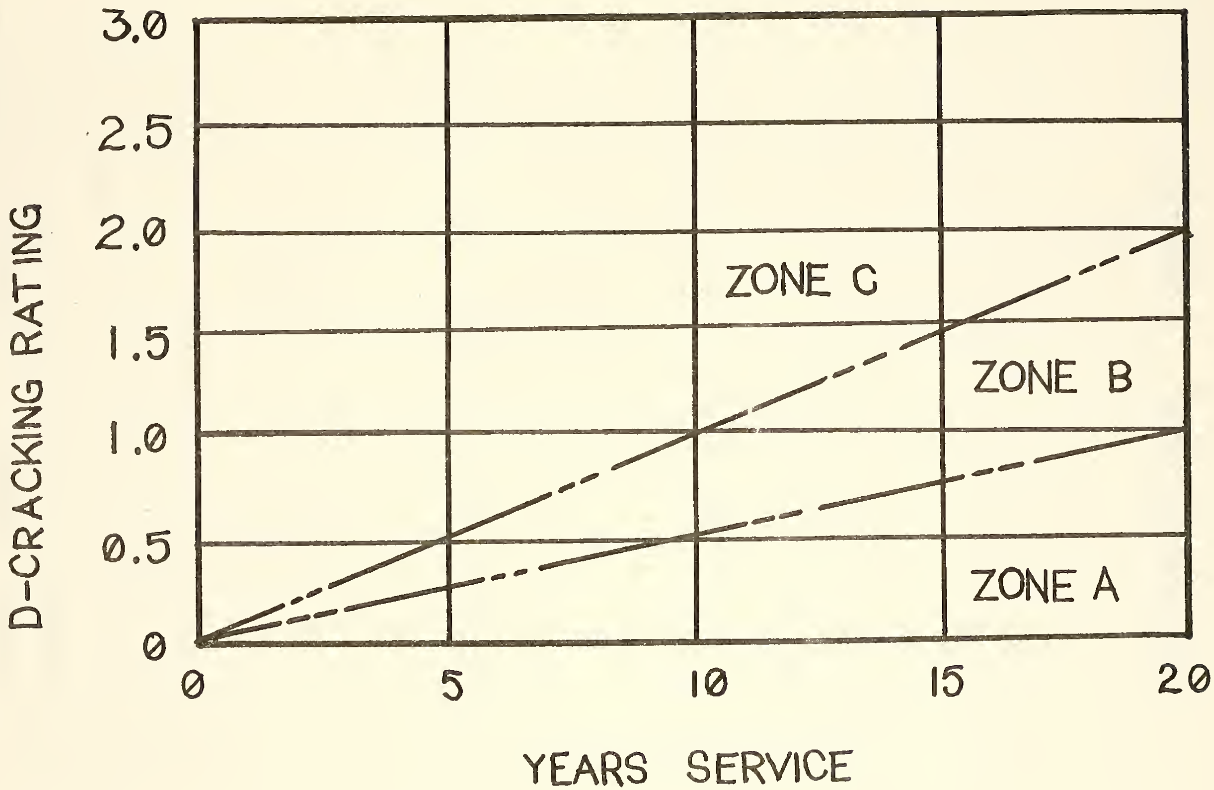

Figure 2. Field Performance Rating

Pore Index	Field Performance Rating
0-25	A
26-35	B
> 35	C

This correlation is for stone samples. The Illinois study showed no apparent correlation for gravels. Further study is being conducted in this area.

The results of the Illinois research indicate that the pore index test is suitable for crushed stone evaluation. Based on their initial results, good performance can be expected from an aggregate with a pore index value smaller than 25. Aggregate with test values in the range of 26 to 35 can be expected to show a moderate level of D-cracking within 20 years, while a value greater than 35 indicates a potential for severe D-cracking within a period of 20 years.

The Illinois DOT made some recommendations based on their results. They suggested that the following criteria might be used for determining the gradation of the aggregate in order to improve the pavement conditions:

Pore Index	Maximum Size
0-25	2" - 1½"
26-35	1½"
> 35	3/4"

Another study of D-cracking was done by the Portland Cement Association titled "D-cracking of Concrete Pavements in Ohio" (4). The major highways in Ohio were rated on a scale devised by the PCA. The scale was from 0 to 5, where a rating of 0 indicated no D-cracking and a rating of 5 indicated an area displaying widespread distress. The numbers 1 through 4 indicated stages of D-cracking between 0 and 5,

the severity of the distress increasing with increasing number. This is a subjective rating scale. The PCA used several photographs to characterize the ratings. These were mounted on a board with a counter placed beside each so that the rater could count the number of joints falling into each category as he drove along the section of pavement being surveyed.

A number indicating the average severity of distress of a particular pavement was calculated using the following formula:

$$\frac{1a + 2b + 3c + 4d + 5e}{a + b + c + d + e} \qquad (2)$$

where the numbers "1" through "5" were possible ratings, and "a" through "e" were the number of joints with each rating. The "0" ratings were not considered; thus, the lowest average rating a pavement could have that showed D-cracking was 1.00. Seventeen percent of all joints surveyed in the Ohio study displayed D-cracking; 2.4 percent of all joints surveyed were in need of immediate repair.

Based on their findings and observations, the PCA made the following conclusions:

1. The rating system devised is a rapid and reliable method for determining the extent and severity of D-cracking in concrete pavements.

2. The PCA found that the best test for reproducing the relative durability of coarse aggregate in pavements is the rapid freezing and thawing of air-entrained concrete prisms in water, a procedure similar to ASTM C666.

3. Using the rapid freeze-thaw test procedure, the PCA study showed that reducing the maximum aggregate size improved the frost

durability of the coarse aggregate from sources associated with D-cracking.

4. The nature of the pore system of the coarse aggregate seemed to be significant in determining the degree of saturation of the aggregate in concrete.

Statement of Objective

Kaneuji determined a correlation between the frost durability of concrete laboratory specimens and the pore size distribution of the coarse aggregate used. He determined tentative borderlines, as stated earlier, for the performance of coarse aggregate in concrete pavements. The primary purpose of the study reported here was either to verify those borderlines for various types of coarse aggregate, or to determine more accurate borderlines. Also investigated was the applicability of Kaneuji's correlation to pavements using gravel as coarse aggregate. A third objective was to determine how to use the EDF correlation as an acceptance test for coarse aggregate for freeze-thaw durability. This study encompassed the testing of many samples for which there were field performance records. Through comparison of these records to test results, it is hoped that the EDF test will prove to be superior to other durability tests presently being used.

Approach Used in This Study

Aggregate from fifty-two Indiana highway cores was sampled and tested, as were five stone samples from the earlier PCA study. The pavements from which the cores came were rated according to their field performance, using the PCA rating scale previously mentioned. The

aggregates were tested using mercury intrusion and, from the pore size distributions, an EDF value was determined for each kind of aggregate in the core being studied. An "average" EDF value was assigned to each pavement associated with the cores tested. This "average" value was compared with the field performance rating to determine the correlation between the two. A correlation was determined and the borderline between durable and nondurable aggregate was set.

The PCA samples were added to expand the scope of the study. The PCA had done its own study on D-cracking, as discussed earlier, and the five samples used in this study were assumed to be representative of the aggregate used in pavements studied by the PCA.

Estimated average absorptions were determined for the aggregate found in each section of pavement surveyed in this study. These averages were compared to past and present Indiana State Highway Specifications for maximum allowable absorptions, 3% and 5% respectively. From these comparisons it was determined that the EDF values are a good predictive measure of the performance of aggregates in concrete pavements.

EXPERIMENTAL WORK

Sampling of Highways

It was originally decided to sample aggregate from portland cement concrete interstate highways in Indiana that were not overlaid with a bituminous surface. The bituminous layer made it impossible to rate the pavement because the original concrete surface could not be seen. Several pavements were overlaid due to severe D-cracking distress. These areas were not included in this study. Interstate pavements were chosen because information on them is easily obtainable and abundant, and four-lane highways are easier to core. However, the majority of Indiana interstate highways are in good condition, rating 0 on the PCA field performance scale, so it was decided to sample other Indiana highways that exhibited various degrees of distress. This additional sampling was intended to fill in the entire range of the PCA scale of 0 to 5. With the guidance of the Indiana State Highway Commission staff, several suitable locations were chosen from U.S. and state highways in Indiana.

Information on the pavements was obtained from the records kept on file at the Division of Materials and Tests, ISHC. This information included the following: the contract number, the location of the contract, the source of the aggregate, the type of aggregate used (stone, gravel, or slag), the age of the pavement, and the maximum size

of the aggregate. This information, with the exception of the source of the aggregate, is tabulated in Appendix B.

The site of each core was located near a mile marker or other road sign to aid the coring crew. Each core location was marked by spraying a line on the pavement with orange lacquer. The number to be affixed to the core was sprayed alongside the line. Data sheets were filled out to record the highway number, the core number, the mile marker, and the PCA rating. Several photographs were taken at each site for later use. Core request forms were filled out with the information necessary to locate the core sites. The ISHC coring crew went to each location and the cores were drilled, using a diamond drill. The crew marked the appropriate core number on each core.

Each pavement was initially rated at the site for field performance using the PCA rating scale (4). As mentioned earlier, this scale is from 0 (no D-cracking) to 5 (severe D-cracking). After all the cores were cut and prepared for testing, the pavements were re-rated by three observers. The three persons viewed slides of each location and, using the photographs in the PCA report, assigned a rating to the pavement. The three new sets of ratings were averaged to give an average PCA value for each core. These values are tabulated in Appendix A.

A coding system was used for numbering the cores. The route letter is the first character in the code and refers to a logical and unique portion of highway. Figure 3 shows these portions of highway with their appropriate route letter. The next character in the code is the site number, indicating the number of stops made on that particular portion of the highway. These are numbered consecutively from 1 to the number

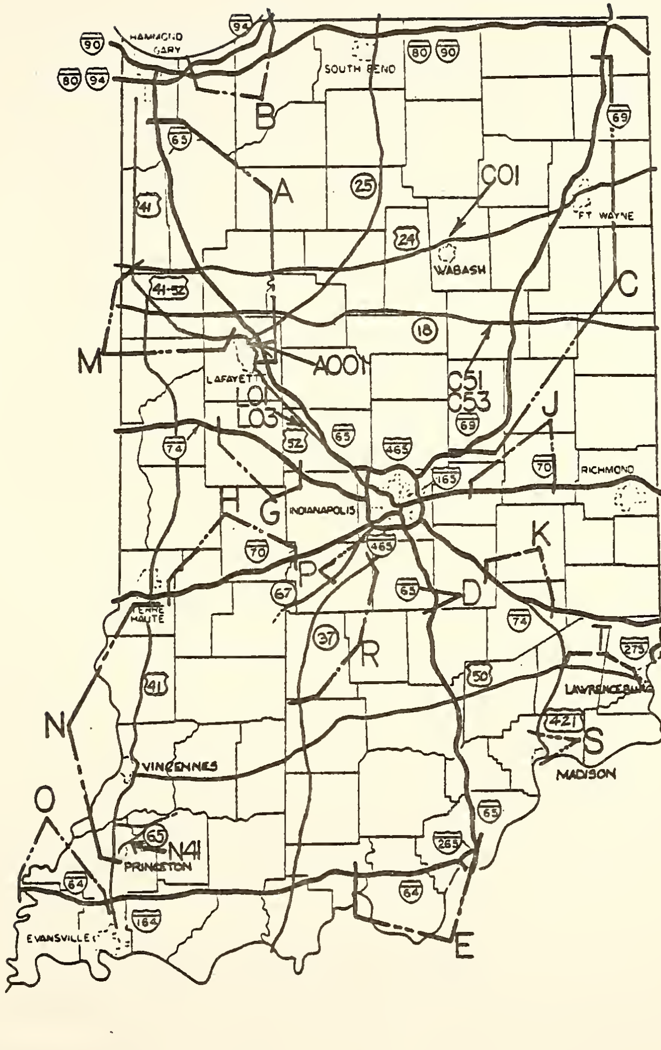

Figure 3. Core Locations

of the last stop on that segment of road. The third character refers to the location of the core at any given site. An odd number signifies a core taken at the edge of the pavement one foot from the shoulder in the traffic lane. An even number indicates a core taken from the center of the traffic lane. Usually two cores were drilled from each contract; in a few instances more than two were drilled.

As mentioned earlier, five stone samples from the PCA were included in this study. Information on these samples can be found in Appendix A and Appendix B.

Preparation of Samples for Testing

Each core was sawed in half longitudinally. One half of the core was crushed into smaller pieces. Crushing was done to facilitate the preparation of samples for mercury intrusion. The other core half was polished. The polished sections were later examined to determine approximate percentages of various rock types present in the pavement, for use in calculating an "average" EDF value for the pavement. Polished section analysis will be discussed later.

The odd-numbered cores (those taken from pavement edges) were the only ones prepared for testing. The even-numbered cores were shelved for possible use at a later time. Eighty-seven cores were prepared for possible testing.

Mercury Intrusion Porosimetry

The method used to measure the pore size distribution of the coarse aggregate was mercury intrusion. The pore diameter range covered by porosimetry is approximately 500 μm to 25 A°. This covers the pore size

range of interest for the purposes of this study. Pores smaller than 25 A^{o} are unimportant because the water in them doesn't freeze at ordinary freezing temperatures.

Theoretical Background

The surface tension of a non-wetting liquid (a liquid that forms a contact angle of at least ninety degrees with a solid) prevents it from entering a pore. This phenomenon is called capillary depression. By applying pressure to the liquid, this opposition can be overcome, and the liquid will enter the pore. The pressure required to force a non-wetting liquid into a pore is a function of the surface properties of the liquid and solid involved, and of the pore geometry. Assuming a cylindrical pore, the relationship is:

$$p = \frac{-4\gamma\cos\Theta}{d} \tag{3}$$

where: p = required external pressure

γ = surface energy of the liquid

Θ = contact angle between the liquid and solid

d = diameter of the pore

The most commonly used liquid is mercury because of its low vapor pressure and its nonreactivity and non-wetting characteristics with most solids. By assuming that the quantity ($-4\gamma\cos\Theta$) is constant for a given solid, the relation between the pressure and the pore diameter becomes a linear inverse. The pore size distribution is then obtained by measuring the volume of intruded mercury at various pressure intervals corresponding to diameters of interest.

In most mercury intrusion studies, the values of θ and γ have been obtained from a handbook of values. This may lead to erroneous results, because the handbook values are general and don't take into account the fact that each material has its own surface properties. Kaneuji measured the intrusion constant $k = (-4\gamma\cos\theta)$ by the method of Winslow and Diamond (5). A sample with drilled holes of known diameter was prepared and the absolute pressure at which these pores were intruded with mercury was measured. The value of k was obtained as the product of the pressure and the diameter of the pores.

$$k = -4\gamma\cos\theta = pd \tag{4}$$

The contact angle θ was calculated by assuming a value of 484 erg/cm^2 for the surface energy, γ, of the mercury. The values of θ were in the range of about 118° to 130°, with an average value of 125°. Therefore, values of γ and θ used in this study were 484 erg/cm^2 and 125°, respectively. The above values were assumed to be suitable for the purposes of this study.

Apparatus

The instrument used was a commercial unit, Catalog No. J5-7125D, American Instrument Company, Silver Spring, Maryland, 20910, modified in some respects by previous users. The maximum capacity of the machine is 60,000 psi. Basically, the porosimeter consists of a pressure vessel, pressure generating pump, and instruments for measuring pressure and intrusion. A fan was added to the side wall of the lower cabinet of the porosimeter to cool the pressure vessel, which becomes warm with increasing pressure. The commercial unit is equipped with two pressure

gauges, 0-5,000 psi and 0-60,000 psi. To increase the accuracy of the readings in the range of pore diameters from 10 μm and 0.5 μm, a 0-500 psi gauge was added. A horizontal filling device was used for measuring the intrusion below one atmosphere of pressure, which corresponds to pore diameters of about 500 μm and 10 μm.

The porosimeter can measure intruded volumes up to a maximum of 0.2 ml. The amount of sample having approximately this pore volume varies with the porosity of the sample. The range of sample masses used in an individual test was about 0.5 gram to 3.5 grams.

Correction of Intrusion Data

Two kinds of corrections are necessary:

1. Below atmospheric pressure, continued compression of air left in the penetrometer after evacuation should be subtracted from each intrusion reading. This value can be calculated using Boyle's Law.

2. High pressure corrections are more complicated. As the pressure is increased, the mercury is compressed and its volume decreases. The pressure also generates heat and thus causes expansion of the mercury. The temperature of the system changes as the test proceeds. Blank tests (tests with a nonporous sample) were made to determine the necessary high pressure corrections.

The pressure must be held at each interval until intrusion ceases. The time required for intrusion depends on the pore characteristics of the sample. Kaneuji (1) found that holding the pressure for one minute was sufficient for the materials being studied. Originally, eighteen pressure intervals above atmospheric pressure were chosen, as shown in Table 1. The correction factors were determined, step-by-step, stopping

Table 1. Correction Factors in Mercury Porosimetry

Pressure (psi)	Correction (ml/ml of Hg in penetrometer)
50	0.00000
100	0.00001
150	0.00003
200	0.00005
300	0.00007
500	0.00009
700	0.00010
1000	0.00012
1500	0.00015
2000	0.00018
3000	0.00023
5000	0.00029
7000	0.00034
10000	0.00040
15000	0.00047
30000	0.00061
45000	0.00067
60000	0.00076

Revised Pressure Intervals and Correction Factors

Pressure (psi)	Correction (ml/ml of Hg in penetrometer)
30	0.00000
50	0.00000
90	0.00002
150	0.00003
250	0.00005
450	0.00008
800	0.00011
1300	0.00015
2300	0.00020
4000	0.00026
7000	0.00034
12000	0.00042
20000	0.00051
35000	0.00063
60000	0.00076

at each chosen interval and measuring the intrusion. As an aide to plotting the pore size distributions, these intervals were later changed and decreased to the fifteen also shown in Table 1. The new intervals give approximately equal spacing of the diameters of interest on a logarithmic scale. Because the volume increase due to heating is greater than the decrease due to compression, the corrections should be added to the intrusion readings.

Preparation of Samples for Intrusion

Aggregate from fifty-two cores and the five PCA samples were tested using mercury intrusion. The remaining thirty-five odd-numbered cores were not tested because they were all rated 0 on the PCA scale, and sufficient cores of this rating were already included in the study. The samples tested were divided into three categories: stone, gravel, and slag. A representative sampling of the aggregate from each core was taken from the crushed material and prepared for intrusion. Samples that were too large to fit into the porosimeter were broken into smaller pieces. Any mortar adhering to the samples was removed using a hammer, chisel, and file. The samples were washed and placed in an oven to dry. They were stored in the oven until testing.

Stone Samples

Twenty-one cores having crushed stone aggregates were tested, as well as the five PCA samples of stone. Most of the cores having stone aggregates contained either one or two types of stone, e.g. one core may have both dolomite and brown limestone while another core may consist entirely of a grey limestone. About 2.0 to 3.0 grams of each kind of

stone were tested. Several individual pieces were tested at one time. This was done twice for each kind of stone in order to obtain a good "average" pore size distribution. The intrusion data were then used to determine EDF values for each sample tested. Each stone type was given a description, using a guide to rocks and minerals (6). The descriptions and calculated EDF values are given for each sample in Appendix A. The pore size distributions are in Appendix C.

Gravel Samples

Aggregate from twenty-one cores from pavements using gravel and seven pavements containing both stone and gravel were tested. As is common with gravel, several different rock types were identifiable within one core. One test was done on each kind of aggregate thought to be representative of the coarse aggregate used in the pavement. On the average, most cores contained six different kinds of rock. Descriptions of each type were made in the same way as for the stone samples. EDF values were calculated from the intrusion data. Descriptions and EDF values are in Appendix A and the pore size distributions are in Appendix C.

Slag Samples

Three cores from pavements with slag aggregates were tested. It is known that slag is an aggregate that is not subject to D-cracking, but tests run on the slag samples indicate a wide range of EDF values, from nondurable to durable. It was decided to determine whether slags fit the correlation equation. Several tests were run on individual pieces of one "type" of slag (slags were differentiated by color and

visible porosity), and the EDF values were calculated. This was done for several "types" of slags. The descriptions and EDF values are tabulated in Appendix A and the pore size distributions are in Appendix C.

Polished Section Analysis

A polished section analysis was used to determine the approximate amounts of each kind of aggregate present in each core. The assumption was made that every aggregate piece measuring one half inch in diameter or greater should have equal significance in determining the numerical percent present in the core. A tally was taken of the number of rocks of each different rock type (corresponding to samples tested) present in the polished section. It was assumed that the polished section was typical of the actual amounts of each type of rock present in the pavement. The percentage of each aggregate type per total number of aggregate pieces counted was calculated for each aggregate type in a polished section.

The method used for determining the percentages of rock types in a pavement is crude, but it presumably gives reasonably accurate results. Other methods for calculating amounts present, such as the linear traverse method or a detailed quantitative analysis, are time-consuming and probably not that much more accurate, considering the fact that only one core was used for testing from each site sampled. The method used was quick and easy to do, thus making it a more desirable choice.

RESULTS

The results of this study are a large number of pore size distributions of the rocks from each core. The entire collection of distributions is found in Appendix C, with a few examples given in the following pages. The legend shown for each graph contains the following information for each curve on the graph: the graphing symbol used, the sample number, the mass of the sample, and the EDF value for the sample. The sample number is a combination of the core number (explained previously) and the number of the aggregate tested.

Kaneuji found that the pore size distribution of one portion of a rock might be somewhat different than another portion of the same rock (1). When individual pieces were tested the largest spread in total pore volume was about 20% relative, with a typical spread being about 10%. However, distributions from about five single pieces gave good values for upper and lower bounds of the pore volume, provided one type of rock was used. Kaneuji therefore tested several small pieces of the same kind of aggregate at one time to get an "average" distribution. This same testing procedure was used in this study. To get a better average and to verify homogeneity of a rock type, two tests were run for each sample for cores containing stone, each test using several small pieces of stone. Figure 4 is a representative set of pore size distributions for crushed stone from a core.

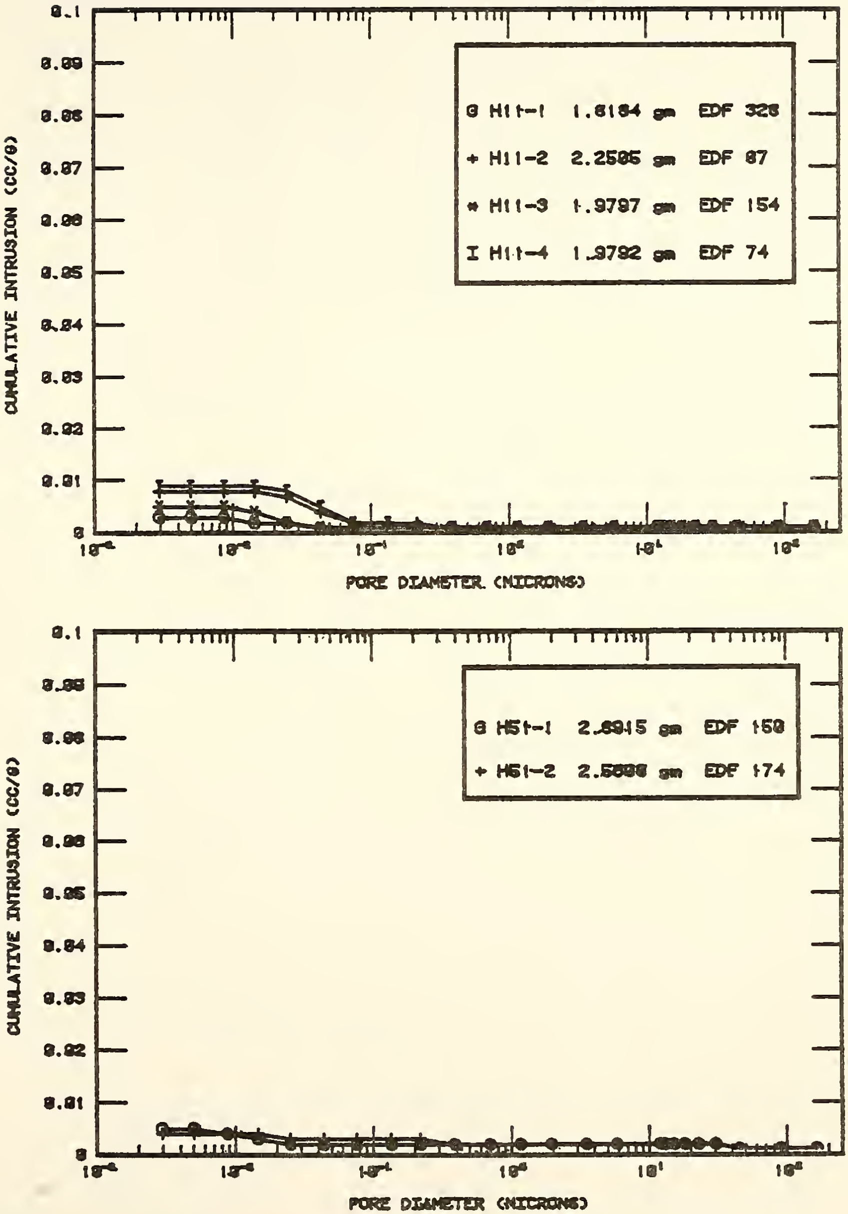

Figure 4. Pore Size Distributions for Stone from Cores H11 and H51

The cores containing gravel were tested somewhat differently. Unlike the stone cores, the gravel cores had several different types of rock present. The average gravel core had about six identifiable kinds of rock and a wide range of EDF values among the different kinds. One test was run on each kind of gravel aggregate, with several pieces of an individual rock type being tested at one time. It was felt that this was sufficient for calculating average EDF values for the gravel aggregates. Figure 5 is a representative set of distributions for gravel aggregate from a core.

The slag samples exhibited a great deal of heterogeneity among the many kinds of slag in each core. Because of this trend it was decided to test several pieces of one kind of slag, one piece at a time. There was considerable spread within any one kind of slag, unlike the stone and gravel samples. Figure 6 shows the pore size distributions of different slags from one core. Figure 7 shows the pore size distributions for slag samples A91-A and A91-B. Sample A91-A consists of four samples of olive-colored slag that were tested to study the range in distributions between samples of the same kind. Sample A91-B is a set of four samples of tan slag, tested for the same purpose as mentioned above.

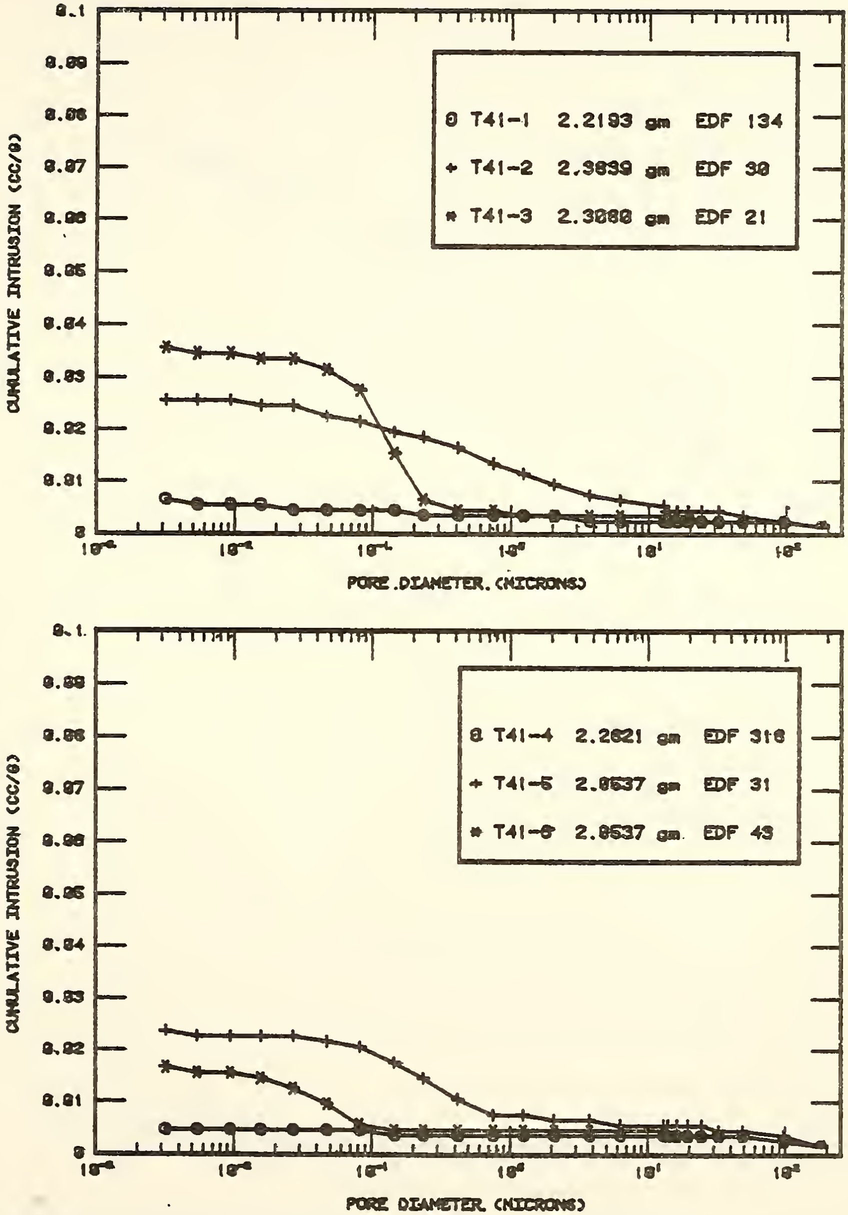

Figure 5. Pore Size Distributions for Gravel from Core T41

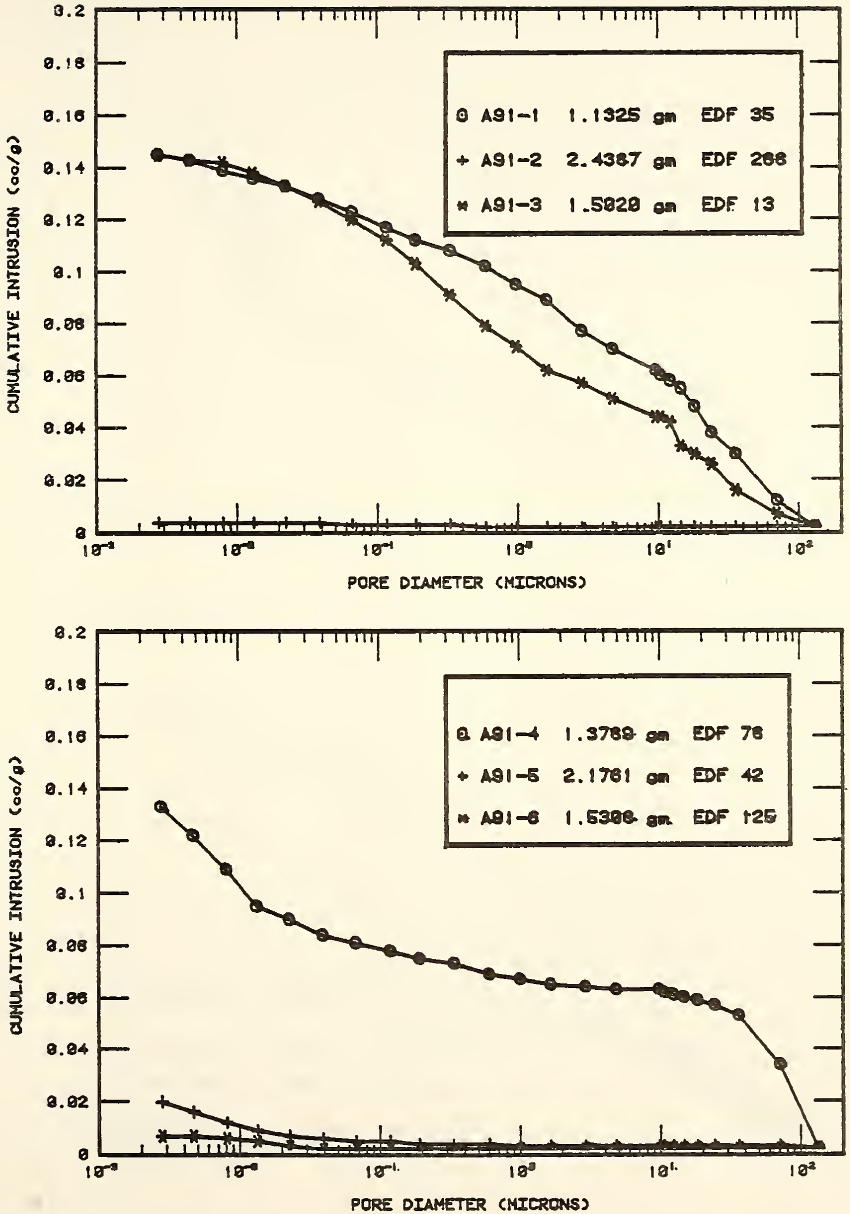

Figure 6. Pore Size Distributions for Slag from Core A91

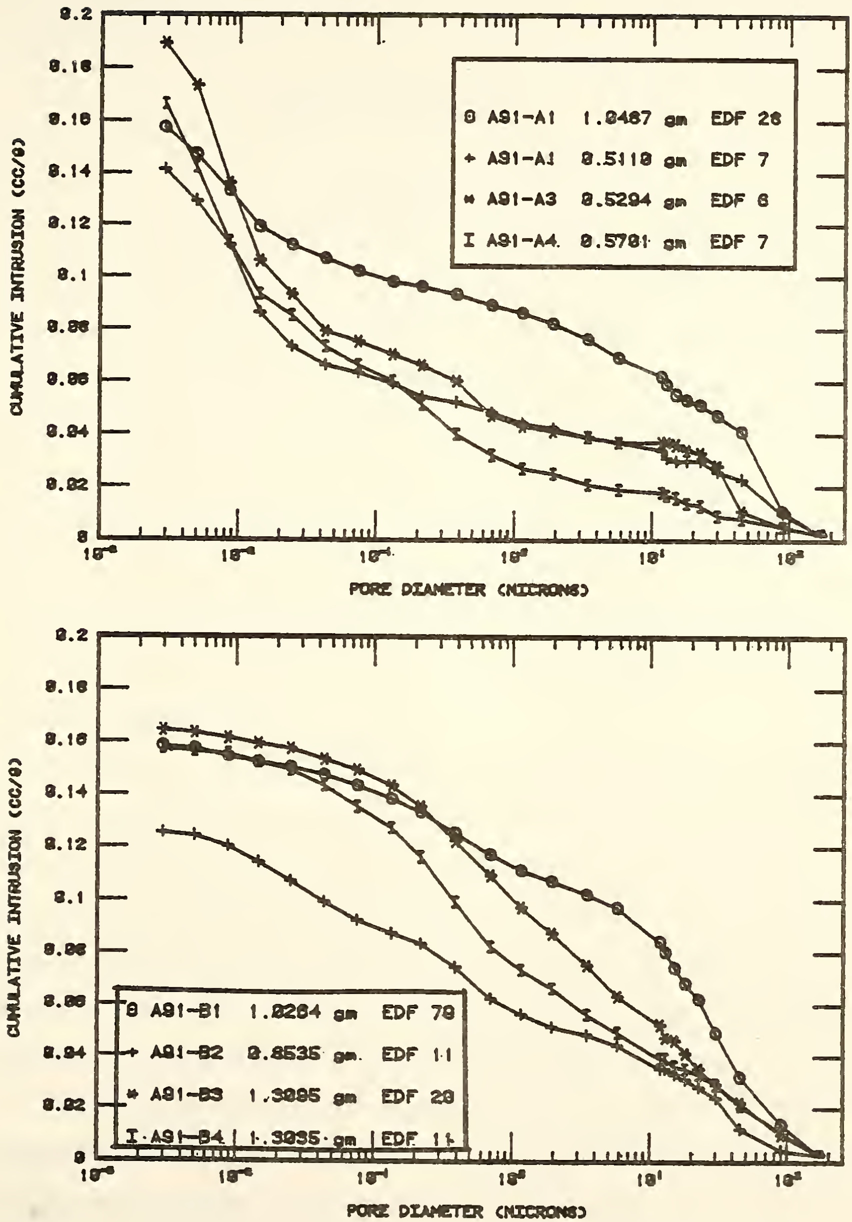

Figure 7. Pore Size Distributions for Slag Samples A91-A and A91-B

DISCUSSION OF RESULTS

Introduction

For the purposes of this study a nondurable aggregate is any aggregate that is likely to cause D-cracking in a pavement. The amount of nondurable aggregate in a pavement is an important factor influencing the performance of a pavement. A pavement can tolerate small amounts of nondurable aggregate, but beyond some maximum allowable amount, the pavement shows D-cracking. The American Society for Testing and Materials, Specification C33 (7), lists a maximum allowable percent of unsound or nondurable material as 18 percent, as measured by the sulfate soundness test. The Indiana State Highway Standard Specifications, 1978 (8), allows a total of 12 percent nondurable aggregate using the sulfate soundness test.

Nondurable can also be defined using the Expected Durability Factor, EDF. As will be discussed subsequently, a value of 50 was chosen as the borderline EDF value — an EDF value greater than 50 means the aggregate is durable and below 50 indicates nondurability. The value of 50 was decided upon based on the field performance of the pavements studied. Using this criterion, the approximate percentage of nondurable aggregate can be determined from the polished section analyses.

In order to correlate the pavement field performance with the expected durability, calculated from the values of the median pore diameter and the pore volume, an "average" EDF was needed. It was easy

to calculate an average value for the stone core samples containing only one kind of rock; an equally weighted average was used. For cores containing two or more kinds of rock, obtaining an "average" value was more complicated. Three approaches were considered for this study. The first method was to use the median value of all of the EDF values associated with one core. However, an average of this type would not always take into account the effect of the nondurable aggregate, as Example 1, on the following page, shows. The second method was similar to an average-by-number method. The percent of each type of rock was known from the polished section analysis so a weighted average by number could be calculated, assuming the percent of each type to be representative of the number of that type of rock present. This method was also insensitive to the nondurable aggregate. The third method was to calculate the numerical average of the EDF values for the durable portion of the aggregate and for the nondurable portion. If a small amount of nondurable aggregate was present, its effects could be neglected and the EDF value for the durable portion would be used; if there were detrimental amounts of nondurable aggregate, the EDF value for this portion would be used. It was this third approach that was used in this study.

The third method involved two decisions: what EDF value is the dividing line between durable and nondurable aggregate and what percentage of nondurable aggregate present in the core is detrimental? The choices were made by letting both values vary until a good correlation was reached, as will be shown later. The decision was crucial to the third method because this value could determine which way the results would turn — durable or nondurable.

The following example will serve to illustrate the differences among the three approaches.

Method I

Rock Type	% Present	EDF	Average EDF
1	10	10	
2	20	20	$\frac{1230}{4} = 307$
3	60	200	
4	10	1000	
	100%	1230	

Method II

Rock Type	Fraction Present	EDF	EDF x Fraction	Average EDF
1	0.10	10	1	
2	0.20	20	4	225
3	0.60	200	120	
4	0.10	1000	100	
	1.00		225	

Method III

Rock Type	% Present	EDF	Average EDF
1	10	10	Nondurable Average = 15
2	20	20	
3	60	200	Durable Average = 600
4	10	1000	

The averages shown above for the nondurable portions were based on non-durable aggregate being defined as having an EDF value of 50 or less for more than ten percent of the aggregate. The median EDF value for the above example is 307, calculated using Method I. This value probably does not give a realistic view of the effect of the thirty percent of nondurable aggregate present. The weighted average by number (% present),

Method II, is 225, still indicating a durable sample in spite of the thirty percent nondurable aggregate. For Method III, the average EDF value of the thirty percent nondurable portion of the sample is 15 and the average of the durable portion is 600. Because of the relatively large amount of nondurable aggregate present, the average EDF value of the nondurable portion may be more representative. It was found that when there is a detrimental amount of nondurable aggregate, the EDF value of the durable portion of the aggregate no longer matters; the pavement will show distress because of the nondurable aggregate. A reasonable starting point for the maximum allowable amount of nondurable aggregate seemed to be ten to fifteen percent. This range seemed reasonable when compared with the limits of 12% and 18% mentioned previously.

A graph of the PCA rating versus the age of the pavement is shown in Figure 8. There doesn't appear to be any correlation with age except to say that pavements less than about seven years of age don't show distress and pavements between the ages of seven and twenty-five years show varying degrees of distress. Pavements older than twenty-five years, that are in distress, are generally overlaid with a bituminous surface, and are therefore eliminated from the study.

The correlation between the PCA rating and the "average" EDF value of the core was determined by plotting the PCA rating versus the average EDF value of the significant portion of the aggregate. Figure 9 is an example of such a plot. The borderline EDF value was a shifting line between EDF values of 40 and 50. The allowable percent of nondurable aggregate was shifted between 10% and 15%. A borderline value

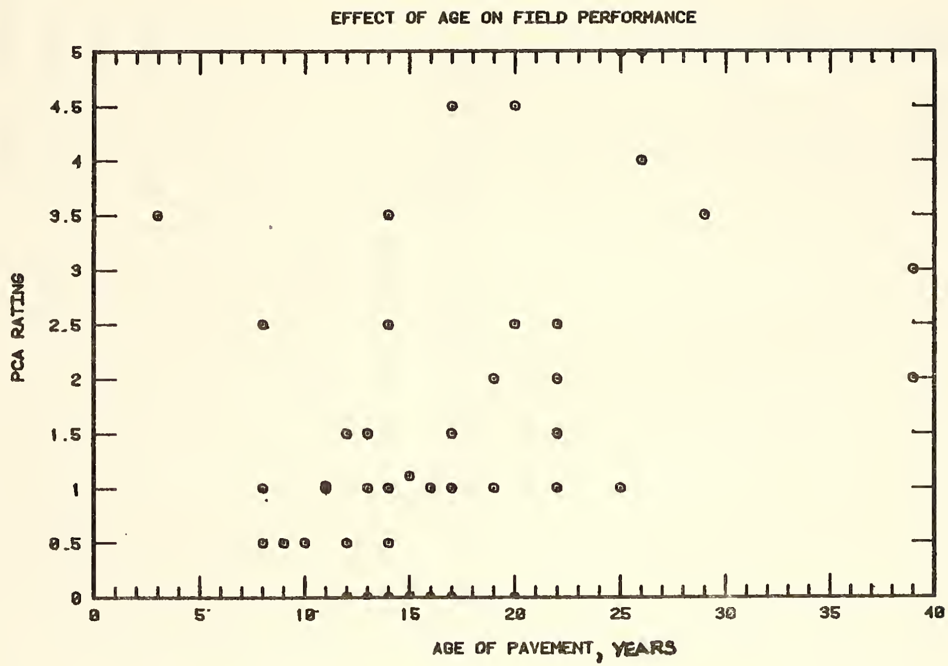

Figure 8. PCA Rating Versus Age of the Pavement

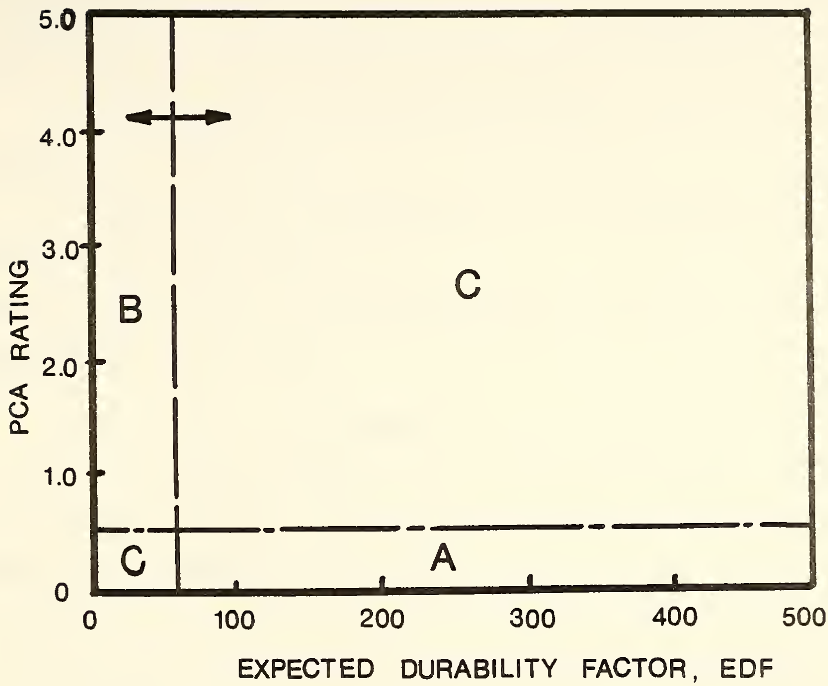

Figure 9. Example Plot

of 0.5 was chosen for the PCA rating to reflect the subjectivity of the pavement evaluation. Figure 9 depicts the correlation between the PCA rating and the "average" EDF value. Using Figure 9 as an example, a perfect correlation would be one in which there were no points plotted in the sections marked C on the graph, all points associated with durable cores plotted in the section marked A, and the nondurable points plotted in section B. Section A signifies a low PCA rating and a high EDF value; cores with points plotted in section A should be durable.

Section B signifies a high PCA rating and a low EDF value; cores with points plotted in section B should be nondurable. If a point is plotted in section C, it can signify one of the following conditions:

1. A low EDF value indicates nondurability but durability is indicated by a low PCA rating.

2. A high EDF value indicates durability but nondurability is indicated by a high PCA rating.

In subsequent discussions, points plotted in sections marked C will be referred to as Zone C problems.

Stone Samples

Due to the general homogeneity of the stone samples within a core from a pavement using stone, the borderline for durable and nondurable aggregate was tentatively set using these samples. Plots of the PCA rating versus the EDF value were made using several borderlines, as discussed earlier. Plots for durable equal to EDF values greater than 30, 40, 50, 60, and 70 were made for an allowable percent of nondurable aggregate of both 10% and 15%. Durable equal to an EDF value greater than 30 was determined to be too lenient because it allowed several points to be in the durable range, even though the pavements associated with the points were in various stages of distress. EDF values greater than 60 and 70 were too conservative; the use of either of these borderlines put a few points associated with pavements exhibiting good field performance in the nondurable region. This left values of about 40 or 50 as possible borderlines.

The following four conditions were plotted for both the stone and gravel samples:

1. Durable = EDF > 40 for 85% of the aggregate

2. Durable = EDF > 40 for 90% of the aggregate

3. Durable = EDF > 50 for 85% of the aggregate

4. Durable = EDF > 50 for 90% of the aggregate

These are shown in Figures 10, 11, 12, and 13 for the stone samples, with tabulated results in Tables 2 and 3.

Figure 10 is the plot for Condition 1 above. It has four points that are Zone C problems. Cores M01 and M11 both had about 12% nondurable aggregate, based on an EDF value of 40 but their PCA ratings indicated that they both had D-cracking distress. When these two points are plotted on Figure 11, for Condition 2, the cores are no longer Zone C problems. They both had more than 10% nondurable aggregate and so they fell into the nondurable Zone B, where they would seem to belong, from their PCA ratings.

Core R31 and sample PCA-E are also Zone C problems. Core R31 had no nondurable aggregate based on the conditions of 1 and 2 above but its PCA rating of 1.0 indicates that the pavement did show D-cracking distress, possibly due to a lack of proper drainage at the core site. Therefore, for both Figures 10 and 11, Core R31 is a Zone C problem. However, when it is plotted on Figures 12 and 13, for Conditions 3 and 4, it is no longer a problem. Based on an EDF value of 50 or more for durable aggregate, Core R31 has 20% nondurable aggregate. PCA-E, on the other hand, is a Zone C problem for each plot, regardless of the conditions. The PCA samples present a special problem because they were not taken from the actual pavement showing the distress, but were instead from the same quarry as the aggregate used in the pavements

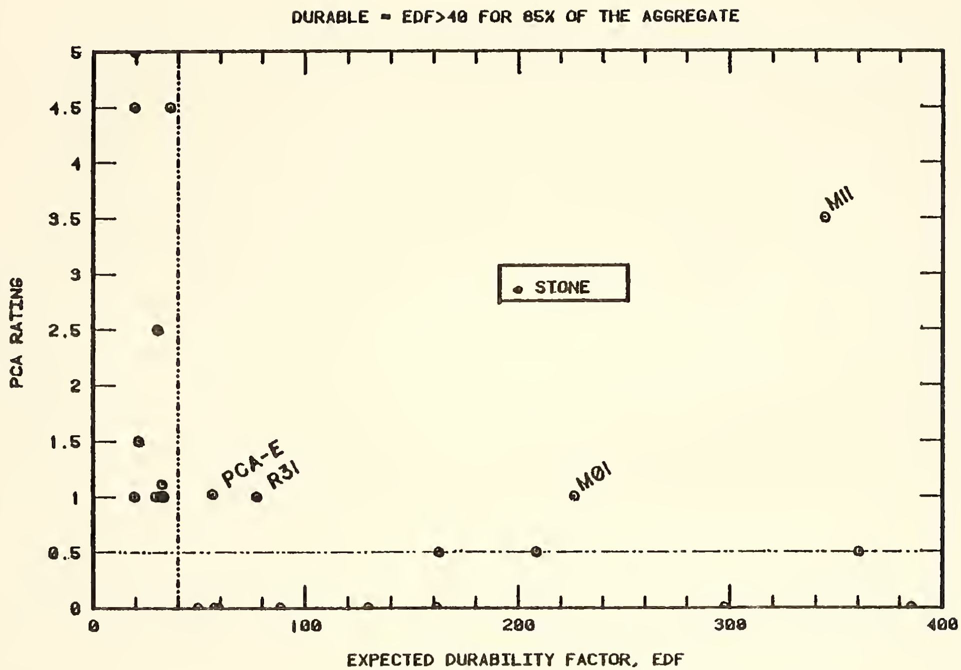

Figure 10. PCA Rating Versus Expected Durability Factor for Stone from Pavement Cores: Condition 1

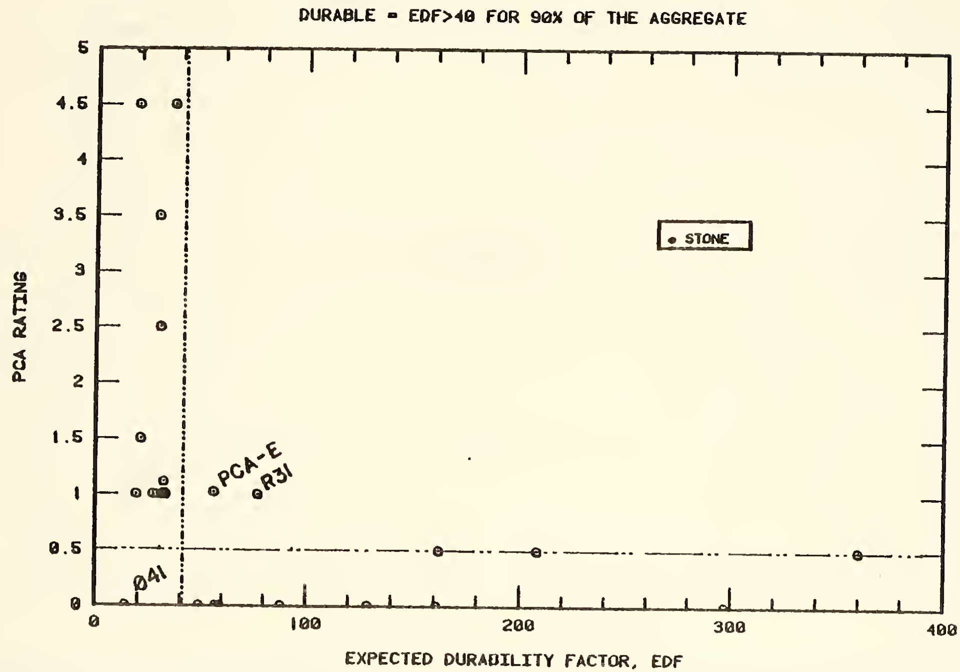

Figure 11. PCA Rating Versus Expected Durability Factor for Stone from Pavement Cores: Condition 2

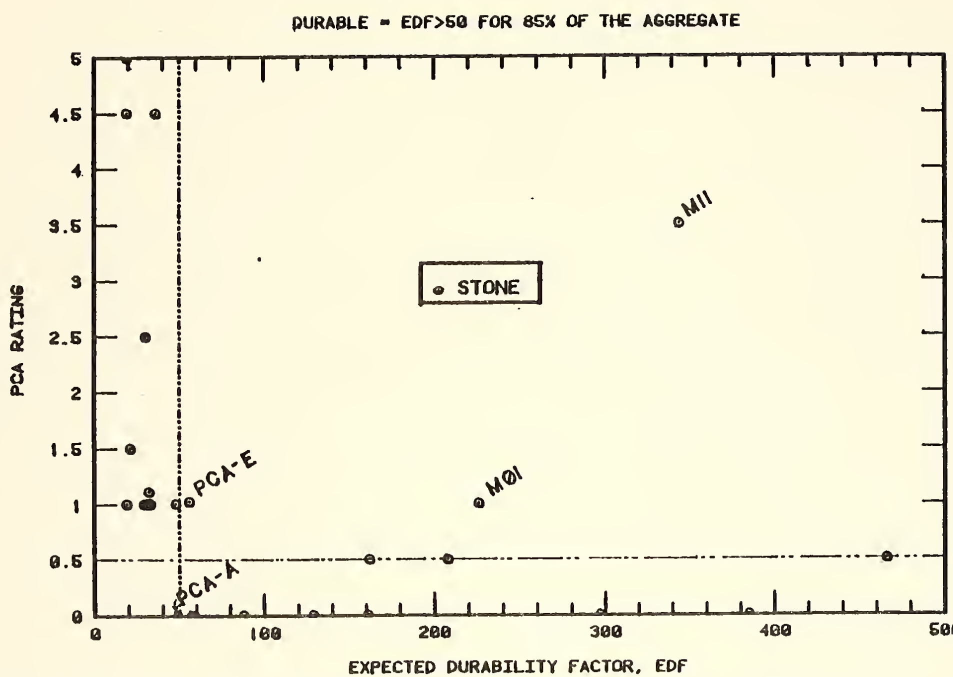

Figure 12. PCA Rating Versus Expected Durability Factor for Stone from Pavement Cores: Condition 3

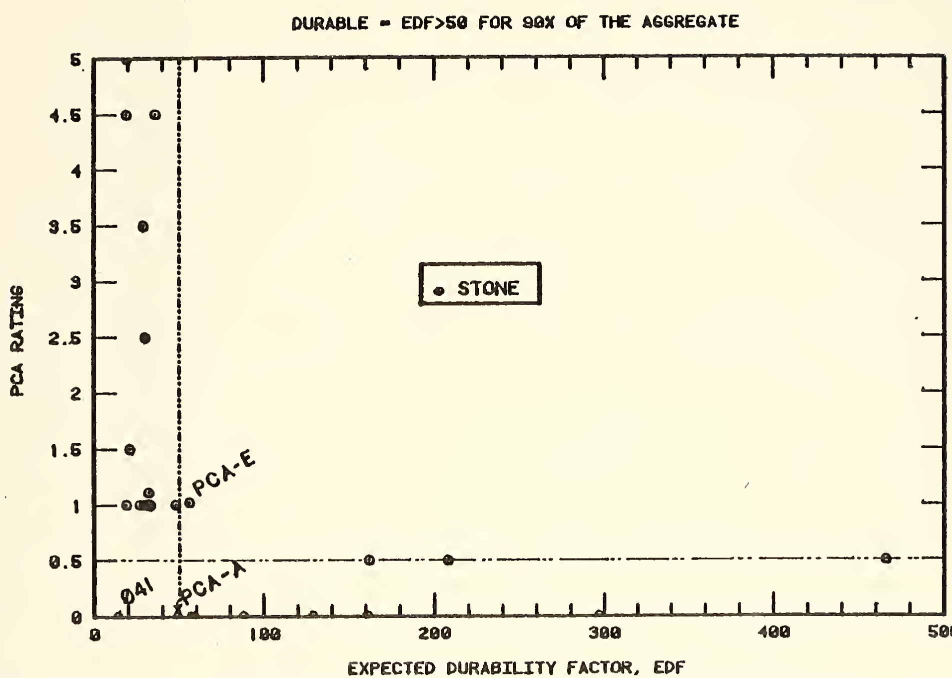

Figure 13. PCA Rating Versus Expected Durability Factor for Stone from Pavement Cores: Condition 4

Table 2. Average Expected Durability Factors For Stone Specimens With Durable = EDF > 40

Core # (Sample #)	Age (Years)	PCA rating	% Nondurable Aggregate	EDF of Nondurable Portion*	EDF of Durable Portion*
A11	12	0.5	0	--	208
A81	15	0.0	0	--	129
B01	13	0.0	0	--	88
C01	20	4.5	100	36	--
C31	16	0.0	--	--	57
C41	15	0.0	--	--	59
C53	17	4.5	100	19	--
C81	16	1.0	88	33	322
C91	17	1.0	100	19	--
E11	9	0.5	5	38	360
G01	17	1.5	36	21	59
H11	12	0.0	0	--	161
H51	14	0.5	0	--	162
M01	14	1.0	12	27	226
M11	14	3.5	12	29	344
M21	20	2.5	54	30	202
N31	17	0.0	6	28	297
O41	12	0.0	13	14	385
R11	8	1.0	36	31	265
R31	11	1.0	0	--	77
T03	14	2.5	39	30	52
PCA-A	20	0.0	0	--	49
PCA-B	11	1.0	100	32	--
PCA-C	14	1.0	100	29	--
PCA-D	15	1.11	100	32	--
PCA-E	11	1.02	0	--	56

* The average EDF value for the durable or nondurable portion is an equally weighted average of EDF values for that portion

Table 3. Average Expected Durability Factors For Stone Specimens With Durable = EDF > 50

Core # (Sample #)	Age (Years)	PCA rating	% Nondurable Aggregate	EDF of Nondurable Portion*	EDF of Durable Portion*
A11	12	0.5	0	--	208
A81	15	0.0	0	--	129
B01	13	0.0	0	--	88
C01	20	4.5	100	36	--
C31	16	0.0	--	--	57
C41	15	0.0	--	--	59
C53	17	4.5	100	19	--
C81	16	1.0	88	33	322
C91	17	1.0	100	19	--
E11	9	0.5	8	39	466
G01	17	1.5	36	21	59
H11	12	0.0	0	--	161
H51	14	0.5	0	--	162
M01	14	1.0	12	27	226
M11	14	3.5	12	29	344
M21	20	2.5	54	30	202
N31	17	0.0	6	28	297
O41	12	0.0	13	14	385
R11	8	1.0	36	32	265
R31	11	1.0	20	48	92
T03	14	2.5	39	30	52
PCA-A	20	0.0	100	49	--
PCA-B	11	1.0	100	32	--
PCA-C	14	1.0	100	29	--
PCA-D	15	1.11	100	32	--
PCA-E	11	1.02	0	--	56

* The average EDF value for the durable or nondurable portion is an equally weighted average of EDF values for that portion

surveyed by the PCA study. They were also sampled at a different time than the quarrying of the road materials. For the purposes of this study the PCA samples were assumed to be representative of the same aggregate used in the pavement when in fact, there could be considerable differences owing to variation in the quarry. Therefore, this must be taken into consideration when evaluating the PCA samples and how they fit the correlation.

Core 041, which had about 13% nondurable aggregate, falls into the expected zone for Condition 1. It becomes a Zone C problem, however, for Condition 2, where only 10% nondurable is allowed. It had a PCA rating of 0 and yet the average EDF value of its nondurable aggregate puts it in the nondurable area. The same situation occurs when imposing Conditions 3 and 4 on the core. A possible explanation of this behavior is that the polished section analysis may not always be an accurate indicator of the actual proportions of different kinds of aggregate present in the pavement. It was assumed that one half core was representative of the entire pavement, which is a broad generalization.

Figures 12 and 13 are the plots for Conditions 3 and 4, respectively. Cores M01 and M11 are Zone C problems for an allowable of 15% nondurable aggregate but are no longer a problem for an allowable of 10%. Sample PCA-E is a Zone C problem, as stated above. Two new problems arise for Conditions 3 and 4. Sample PCA-A is on the borderline between durable and nondurable aggregate with a PCA rating of 0 and an average EDF value of 49. Again, it must be understood that the PCA samples are a special case.

Another assumption was made in this study that may affect the EDF values. The contact angle, θ, was assumed to have a value of 125°,

for use in the equation

$$p = \frac{-4\gamma\cos\Theta}{d} \tag{3}$$

for determining the pore diameters of interest. Kaneuji calculated a range of contact angles, from 118° to 130°. Increasing the angle for sample PCA-A from 125° to 130° had the effect of raising the EDF value from 48 to 53, above the borderline for durable aggregate. Lowering the contact angle to 120° lowered the EDF value to 43. It is possible that the actual contact angle for PCA-A is greater than the assumed 125°. The effect of changing the contact angle isn't significant where the aggregates are plainly nondurable because the change in EDF values is only a few percent, not enough to put their EDF values over the borderline.

Of the four conditions plotted in Figures 10 through 13 for stone samples, Figure 13 representing Condition 4, where durable is equal to an EDF value greater than 50 for 90% of the aggregate, seems to be the best combination for predicting the performance of aggregates.

Gravel Samples

The gravel samples were treated in the same manner as the stone samples for determining borderlines. The durable and nondurable portions were determined from polished section analyses (Appendix A) and the average EDF values were calculated for each portion, using the criteria of EDF values greater than 40 and greater than 50. Tables 4 and 5 show tabulated values of percentage of nondurable aggregate and calculated EDF values. Figures 14 through 17 are the plots for Conditions 1 through 4 for the gravel and stone samples. In all these figures there

Table 4. Average Expected Durability Factors For Gravel Specimens With Durable = EDF > 40

Core #	Age (Years)	PCA rating	% Nondurable Aggregate	EDF of Nondurable Portion*	EDF of Durable Portion*
A001	3	3.5	53	18	360
C11	19	2.0	43	28	194
C61	16	0.0	37	28	92
C101	12	0.0	47	22	138
D01	10	0.5	57	28	268
G31	17	1.0	21	36	258
H01	12	0.5	24	15	309
J21	15	0.0	0	--	177
K21	19	1.0	60	30	197
L01	39	2.0	6	20	324
L03	39	3.0	47	17	238
M31	26	5.0	44	37	359
M51	29	3.5	32	25	238
N01	26	4.0	36	16	265
N21	8	2.5	22	29	330
N41	25	1.0	30	28	301
N51	14	1.0	32	9	354
P01	14	0.0	9	26	186
P11	22	1.0	40	24	256
R01	8	0.5	69	27	356
S01	13	1.5	32	22	307
S11	13	1.0	65	22	284
T11	12	1.5	73	32	532
T21	22	1.0	50	14	180
T41	22	1.5	64	27	165
T43	22	1.5	65	24	164
T51	22	2.5	65	20	265
T53	22	2.0	63	25	146

* The average EDF value for the durable or nondurable portion is an equally weighted average of EDF values for that portion

Table 5. Average Expected Durability Factors For Gravel Specimens With Durable = EDF > 50

Core #	Age (Years)	PCA rating	% Nondurable Aggregate	EDF of Nondurable Portion*	EDF of Durable Portion*
A001	3	3.5	53	18	360
C11	19	2.0	43	28	194
C61	16	0.0	57	32	109
C101	12	0.0	53	26	170
D01	10	0.5	57	28	268
G31	17	1.0	35	43	300
H01	12	0.5	35	30	353
J21	15	0.0	6	41	199
K21	19	1.0	60	30	197
L01	39	2.0	6	20	324
L03	39	3.0	47	17	238
M31	26	5.0	44	37	359
M51	29	3.5	32	25	238
N01	26	4.0	39	25	321
N21	8	2.5	22	29	330
N41	25	1.0	30	28	301
N51	14	1.0	32	9	354
P01	14	0.0	9	26	186
P11	22	1.0	65	30	310
R01	8	0.5	69	27	356
S01	13	1.5	32	22	307
S11	13	1.0	65	22	284
T11	12	1.5	73	32	532
T21	22	1.0	57	24	225
T41	22	1.5	74	31	226
T43	22	1.5	65	24	164
T51	22	2.5	75	29	375
T53	22	2.0	63	25	146

* The average EDF value for the durable or nondurable portion is an equally weighted average of EDF values for that portion

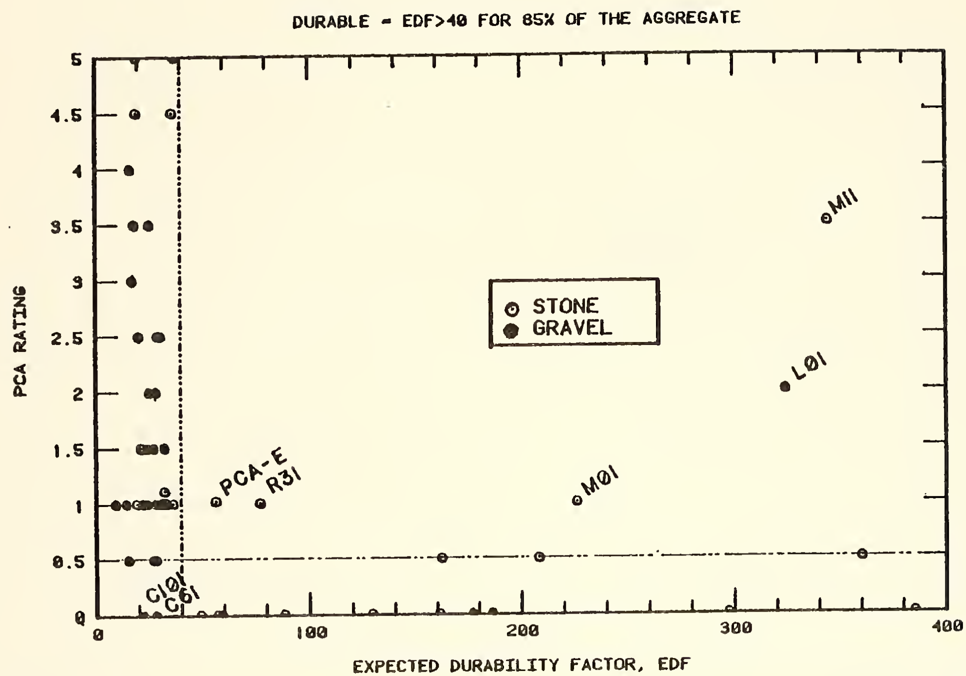

Figure 14. PCA Rating Versus Expected Durability Factor for Gravel and Stone from Cores: Condition 1

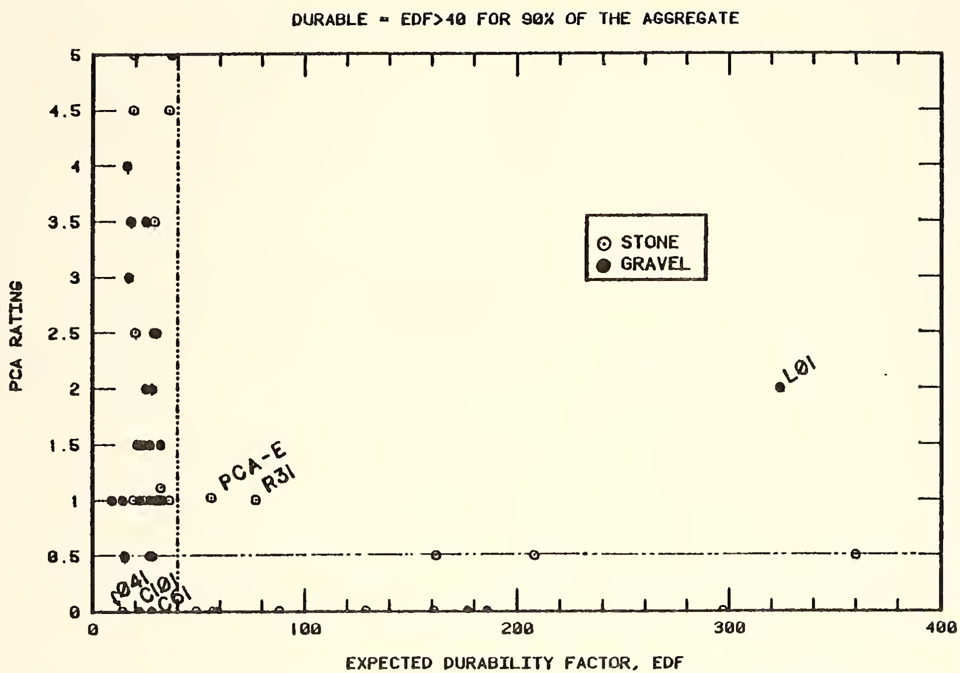

Figure 15. PCA Rating Versus Expected Durability Factor for Gravel and Stone from Cores: Condition 2

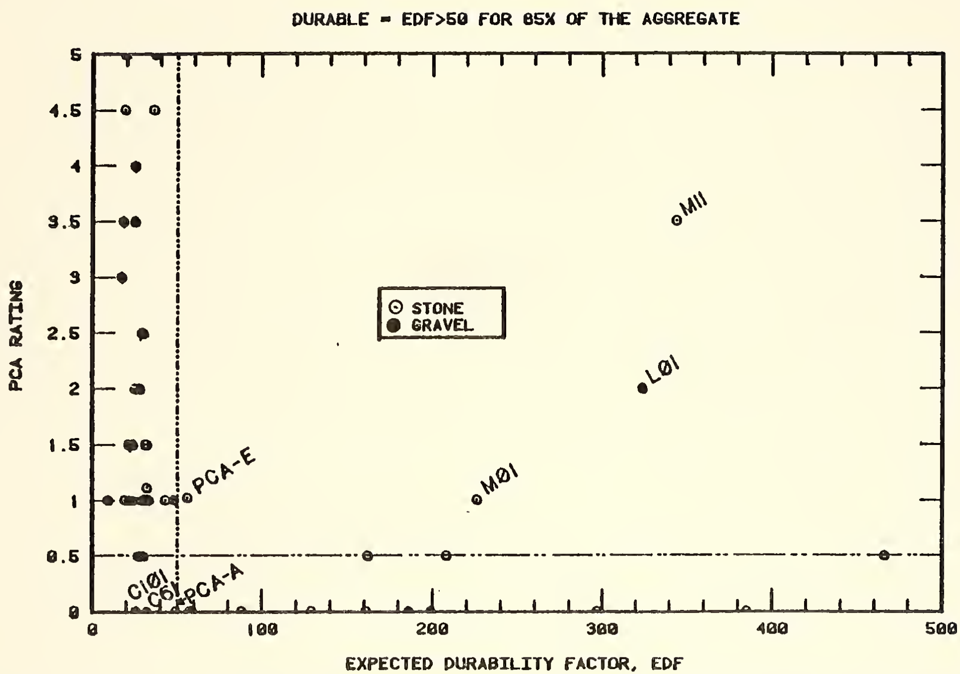

Figure 16. PCA Rating Versus Expected Durability Factor for Gravel and Stone from Cores: Condition 3

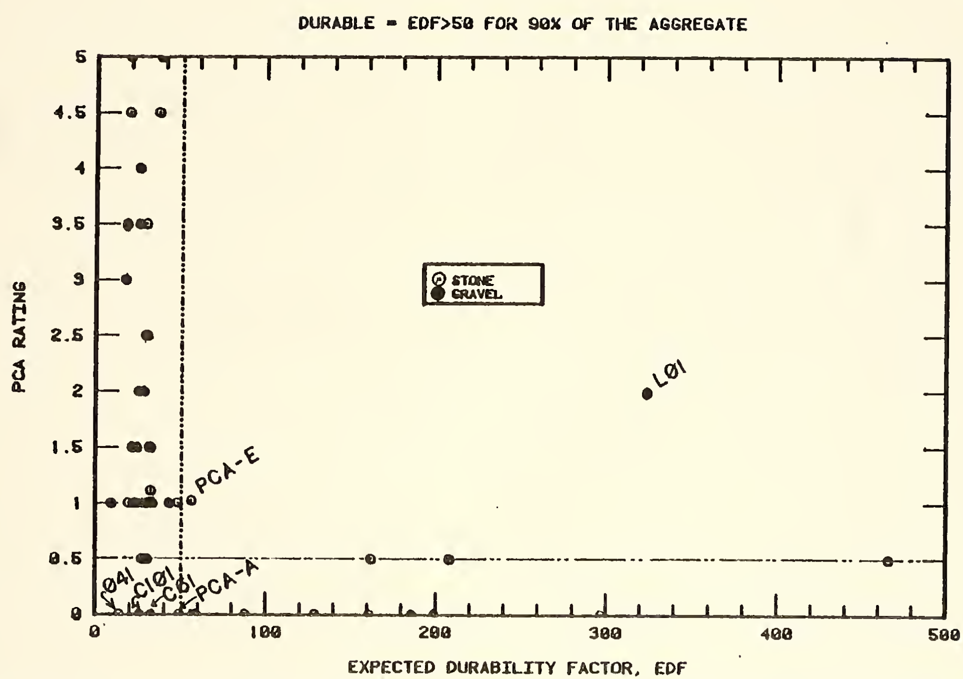

Figure 17. PCA Rating Versus Expected Durability Factor for Gravel and Stone from Cores: Condition 4

are three cores that don't seem to fit the correlation, L01, C101, and C61, besides the previously mentioned stone sample problems. L01 may be explained by the age of the pavement. This particular section of pavement is 39 years old.

Cores C101 and C61 had some highly porous aggregates present, similar to slag in appearance. As will be discussed shortly, the slag samples didn't fit the correlation at all. It is believed that this is due to the great spread in pore sizes. This may explain why C61 and C101 don't fit into the expected trend.

Other than the above-mentioned samples, all of the other gravel samples tested fell within the expected range on the graphs. Based on the graphs, Figures 14 through 17, Condition 4 was again picked as the deciding criterion for determining aggregate performance.

Slag Samples

As was stated earlier, the slag samples didn't fit the correlation. Pavements using slag showed no signs of distress but many of the calculated EDF values were well below 50. The range of pore size distributions was large for any one "kind" of slag, showing a great deal of variation. Therefore, several tests were run on one kind of slag, testing one piece at a time. This would be an indicator of the amount of variation within a sample. Five different kinds of slag were tested in this way. The pore size distributions of all of the slag samples tested are in Appendix C, as well as in the section on Results.

Spread Factors were calculated for each slag sample tested. The Spread Factor, SF, was defined by Kaneuji as the ratio of the 25-percentile pore diameter to the median, or 50-percentile, diameter. For

samples with a large variation in pore size the SF ought to be a large number, whereas most of the stone and gravel aggregates used in pavements examined in this study have low SF values, indicating much less variation in pore size. Table 6 is a list of SF values for all of the slags tested and for several limestones and sandstones of various pore size distributions. It can be seen from this table that 14 of the 16 slag samples have SF values of at least 5 while the majority of the limestone samples have SF values below 5. Kaneuji found that when he included the SF in the correlation, the aggregate samples with large SF values had higher EDF values than samples with about the same total intrusion and median diameter but lower SF values. He didn't pursue this study because the majority of his samples exhibited low SF values and so he didn't have enough spread to warrant inclusion of the SF in the correlation equation. However, based on his observations of laboratory samples and pavements, it is suggested that the large spread in pore size for slags and some highly porous limestones increases the durability of the aggregate. Further study in this area is required in order to understand how to apply the Expected Durability Factor correlation to aggregate with large Spread Factors.

Comparison With 24-Hour Absorption

Absorption is a rough measure of pore volume. Less durable aggregates will generally have higher absorption values, but the absorption isn't always a precise indicator of durability. Kaneuji measured 24-hour absorption for comparison to the total pore volume as measured by mercury intrusion. From his values of 24-hour absorption and values of total intrusion for each of his samples, a ratio of 24-hour

Table 6. Comparison of Spread Factors for Slag and Nonslag Aggregates

Sample #	Type	25 Percentile Diameter, D_{25} (microns)	Median Diameter, D_{50} (microns)	$SF = \frac{D_{25}}{D_{50}}$
A91-A1	olive slag	44.4	3.85	11.52
A91-A2	olive slag	10.3	0.04	245.16
A91-A3	olive slag	0.77	0.03	29.57
A91-A4	olive slag	0.42	0.04	9.74
A91-B1	tan slag	33.3	12.5	2.67
A91-B2	tan slag	16.0	0.61	26.00
A91-B3	tan slag	16.0	0.23	70.00
A91-B4	tan slag	9.40	0.73	12.82
B11-A1	blue-grey slag	0.21	0.02	13.33
B11-A2	blue-grey slag	0.43	0.09	4.80
B11-A3	blue-grey slag	18.8	0.02	1070.59
B11-A4	blue-grey slag	2.50	0.15	16.41
B11-B1	black-grey slag	34.0	1.28	26.60
B11-B2	black-grey slag	82.8	24.6	3.37
B11-B3	black-grey slag	25.8	1.77	14.52
B11-B4	black-grey slag	35.5	1.45	24.44
C01-1	limestone	0.08	0.05	1.66
C61-1	porous yellow	26.6	3.40	7.83
C101-1	porous amber limestone	24.6	9.68	2.54
C101-2	porous pink limestone	45.6	22.8	2.00
C101-3	porous grey limestone	19.7	3.85	5.12
G01-1	brown limestone	1.45	0.09	16.36
H51-1	limestone	0.02	0.01	1.71
M11-3	tan limestone	1.23	0.40	3.08
N01-3	rust limestone	1.03	0.80	1.29
N41-2	porous brown limestone	42.0	10.6	3.95
N41-4	grey sandstone	1.95	1.37	1.43
N51-2	orange limestone	0.48	0.18	2.67
O41-4	brown limestone	0.02	0.008	2.13
P01-3	yellow limestone	5.24	2.90	1.80
R11-3	brown/orange limestone	2.80	1.92	1.46
T03-4	grey limestone	8.87	0.29	31.11

absorption to total intrusion was calculated for each sample tested in Kaneuji's study. An average value of 84.49% was calculated for the ratio. By multiplying the total pore volume of each sample run in this study by this ratio, an estimated 24-hour absorption was determined for each sample. An average estimated 24-hour absorption was determined for each set of samples from a core. The estimated 24-hour absorption for each sample in a core was multiplied by the fraction of that type of rock present in the core. All of the values were then summed to determine the average estimated 24-hour absorption for the core. Table 7 is a tabulation of the average estimated absorption values for all cores tested. Table 8 is a list of only the values associated with cores that didn't pass the criteria of an EDF value greater than 50 for 90% of the aggregate.

Core C41, listed in Table 7, passes the 5% specification and the EDF test criteria, but it fails the 3% specification. The aggregate from C41 was coarse-pored, which may account for its durability. The EDF test considers more than just the pore volume; it also takes the pore diameter into consideration. Therefore, aggregates that have a large pore volume and a high amount of fine pores will be nondurable; aggregates that have a large pore volume and large pore diameters will be durable, which is the case with Core C41.

It can be seen from Table 8 that the present Indiana State Specification of a maximum allowable absorption of 5% would allow all of the aggregates associated with D-cracking to pass. The 3% maximum would allow 35 out of 37 to pass. However, the EDF test only let one sample pass that showed distress. The sample that passed is PCA-E, one of the

Table 7. Estimated 24 Hour Absorptions

Core No. (Sample No.)	Average Total Intrusion (cc/g)	Estimated 24 Hour Absorption,%	Passes ≤ 5% Specif.?	Passes ≤ 3% Specif.?	Passes EDF Test?
PCA-A	0.090	7.57	No	No	No
PCA-B	0.034	2.83	Yes	Yes	No
PCA-C	0.030	2.50	Yes	Yes	No
PCA-D	0.036	1.69	Yes	Yes	No
PCA-E	0.011	0.93	Yes	Yes	Yes
A001	0.022	1.83	Yes	Yes	No
A11	0.007	0.60	Yes	Yes	Yes
A81	0.008	0.69	Yes	Yes	Yes
B01	0.010	0.81	Yes	Yes	Yes
C01	0.018	1.53	Yes	Yes	No
C11	0.016	1.34	Yes	Yes	No
C31	0.022	1.89	Yes	Yes	Yes
C41	0.042	3.53	Yes	No	Yes
C53	0.049	4.13	Yes	No	No
C61	0.020	1.73	Yes	Yes	No
C81	0.022	1.84	Yes	Yes	No
C91	0.042	3.57	Yes	No	No
C101	0.036	3.02	Yes	No	No
D01	0.024	2.01	Yes	Yes	No
E11	0.005	0.41	Yes	Yes	Yes
G01	0.019	1.62	Yes	Yes	No
G31	0.008	0.71	Yes	Yes	No
H01	0.016	1.38	Yes	Yes	No
H11	0.004	0.31	Yes	Yes	Yes
H51	0.004	0.30	Yes	Yes	Yes
J21	0.009	0.80	Yes	Yes	Yes
K21	0.017	1.43	Yes	Yes	No
M01	0.009	0.78	Yes	Yes	No
M11	0.010	0.86	Yes	Yes	No
M21	0.015	1.23	Yes	Yes	No
M31	0.010	0.88	Yes	Yes	No
M51	0.015	1.23	Yes	Yes	No
N01	0.021	1.77	Yes	Yes	No
N21	0.009	0.80	Yes	Yes	No
N31	0.004	0.33	Yes	Yes	Yes
N41	0.015	1.26	Yes	Yes	No
N51	0.035	2.96	Yes	Yes	No
O41	0.009	0.80	Yes	Yes	No
P01	0.015	1.23	Yes	Yes	Yes
P11	0.018	1.53	Yes	Yes	No

Table 7. Continued

Core No. (Sample No.)	Average Total Intrusion (cc/g)	Estimated 24 Hour Absorption,%	Passes $\leq$ 5% Specif.?	Passes $\leq$ 3% Specif.?	Passes EDF Test?
R01	0.022	1.86	Yes	Yes	No
R11	0.015	1.29	Yes	Yes	No
R31	0.011	0.95	Yes	Yes	No
S01	0.015	1.24	Yes	Yes	No
S11	0.028	2.36	Yes	Yes	No
T03	0.016	1.39	Yes	Yes	No
T11	0.018	1.56	Yes	Yes	No
T21	0.033	2.75	Yes	Yes	No
T41	0.021	1.79	Yes	Yes	No
T43	0.028	2.37	Yes	Yes	No
T51	0.027	2.24	Yes	Yes	No
T53	0.016	1.37	Yes	Yes	No

Table 8. Estimated 24 Hour Absorptions for Pavements Showing Signs of Distress

Core No. (Sample No.)	Shows D-cracking *	Estimated 24 Hour Absorption,%	Passes ≤ 5% Specif.?	Passes ≤ 3% Specif.?	Passes EDF Test?
PCA-B	Yes	2.83	Yes	Yes	No
PCA-C	Yes	2.50	Yes	Yes	No
PCA-D	Yes	1.69	Yes	Yes	No
PCA-E	Yes	0.93	Yes	Yes	Yes
A001	Yes	1.83	Yes	Yes	No
C01	Yes	1.53	Yes	Yes	No
C11	Yes	1.34	Yes	Yes	No
C53	Yes	4.13	Yes	No	No
C81	Yes	1.84	Yes	Yes	No
C91	Yes	3.57	Yes	No	No
D01	?	2.01	Yes	Yes	No
G01	Yes	1.62	Yes	Yes	No
G31	Yes	0.71	Yes	Yes	No
H01	?	1.38	Yes	Yes	No
K21	Yes	1.43	Yes	Yes	No
M01	Yes	0.78	Yes	Yes	No
M11	Yes	0.86	Yes	Yes	No
M21	Yes	1.23	Yes	Yes	No
M31	Yes	0.88	Yes	Yes	No
M51	Yes	1.23	Yes	Yes	No
N01	Yes	1.77	Yes	Yes	No
N21	Yes	0.80	Yes	Yes	No
N41	Yes	1.26	Yes	Yes	No
N51	Yes	2.96	Yes	Yes	No
P11	Yes	1.53	Yes	Yes	No
R01	?	1.86	Yes	Yes	No
R11	Yes	1.29	Yes	Yes	No
R31	Yes	0.95	Yes	Yes	No
S01	Yes	1.24	Yes	Yes	No
S11	Yes	2.36	Yes	Yes	No
T03	Yes	1.39	Yes	Yes	No
T11	Yes	1.56	Yes	Yes	No
T21	Yes	2.75	Yes	Yes	No
T41	Yes	1.79	Yes	Yes	No
T43	Yes	2.37	Yes	Yes	No
T51	Yes	2.24	Yes	Yes	No
T53	Yes	1.37	Yes	Yes	No

*A question mark indicates a borderline pavement condition with a PCA rating of 0.5.

Portland Cement Association samples. As mentioned previously, the PCA samples were assumed to be representative of the aggregate used in the pavement, but this may not be the case. Therefore, the PCA rating assigned to the pavement may not actually apply to the sample tested. The three questionable samples, indicated by question marks in the column marked "shows D-cracking" are rated 0.5 on the PCA scale, and so there may or may not be D-cracking.

The above comparison would seem to indicate that a more restrictive test is needed to predict the performance of an aggregate. The EDF test is a more discriminatory test because it considers both the pore volume, PV, and the median pore diameter, MD, of an aggregate. The 24-hour absorption test only considers a partial measure of the pore volume. Therefore, it is suggested that mercury intrusion analysis replace the 24-hour absorption test in predicting the performance of an aggregate.

CONCLUSIONS

1. The Expected Durability Factor, EDF, calculated from the pore size distribution of a coarse aggregate, can be correlated with the observed field performance of pavements made with that aggregate, for the first 30 years of pavement life.

2. The following durable-nondurable borderline for a single aggregate type was found to be the best for stone and gravel aggregate with a maximum size of 1½" to 2½" and a Spread Factor, SF, less than 5:

EDF	Predicted Durability
0-50	Nondurable
> 50	Durable

3. It was determined that greater than 10% nondurable aggregate present in a pavement has a detrimental effect on the pavement. Therefore, the criterion for durable aggregate is an EDF value greater than 50 for 90% or more of the aggregate, when the SF is less than 5.

4. Durability, as predicted by the EDF test, is a more accurate indicator of the durability of the coarse aggregate in a pavement than is 24-hour absorption.

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BIBLIOGRAPHY

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APPENDICES

APPENDIX A

This appendix contains the approximate polished section analyses for each core tested. The data were used in determining the percent of nondurable aggregate present in the pavement. Also contained in this appendix is information concerning the PCA samples from Ohio. The following is a list of the tables in this appendix:

Table A1 contains the core number, the individual sample number, a brief description of the sample tested, the percent of the aggregate sample present in the polished section, the calculated EDF values, and the PCA rating assigned to the pavement from which the sample came.

Table A2 contains the core number and the test (or sample) number for the slag samples tested, and the sample number and test number for the PCA samples. Also found in this table are a brief description of the sample tested, the calculated EDF values, and the PCA rating assigned to the pavement using the aggregate tested.

Table A1. Polished Section Analyses for Cores of Pavements Using Stone or Gravel

Core No.	Sample No.	Description of Sample	% Present in Core	EDF Value	PCA Rating of Pavement
A001	1	grey granite	18	562	3.5
	2	mica	20	233	
	3	limestone w/orange flecks	2	68	
	4	orange sandstone	2	12	
	5	tan limestone	51	25	
	6	pink granite	7	493	
A11	1,2	dolomite	100	216,200	0.5
A81	1,3	dark brown sandstone	80	63,69	0
	2,4	rust sandstone	20	185,198	
B01	1,2	dolomite w/traces of oil	100	84,92	0
C01	1,2	limestone	100	34,37	4.5
C11	1	quartz	13	206	2.0
	2	dark grey limestone	44	181	
	3	grey/brown limestone	17	25	
	4	yellow limestone	4	31	
	5	tan limestone	22	29	
C31	1	All aggregate tested		93	0
	2	for core C31 was		26	
	3	a combination of	100	74	
	4	brown and grey		55	
	5	limestone.		35	
C41	1	All aggregate tested		157	0
	2	for core C41 was a com-	100	25	
	3	bination of amber and		31	
	4	white limestone.		56	
	5			28	
C53	1,2	white limestone	59	21,24	4.5
	3,4	tan limestone	41	14,15	
C61	1	porous pale yellow limestone	20	41	0
	2	coarse white limestone	2	56	
	3	grey/tan limestone	16	25	
	4	yellow limestone	21	30	
	5	black slate	19	213	
	6	tan/grey sandstone	22	59	

Table A1. Continued

Core No.	Sample No.	Description of Sample	% Present in core	EDF Value	PCA Rating of Pavement
C81	1	pale yellow limestone	18	21	1.0
	4	pale grey limestone	32	24	
	2,5	black slate	12	322,353	
	3,6	brown sandstone	34	38,50	
	7	dark brown siltstone	4	30	
C91	1	All aggregate tested for	100	16	1.0
	2	core C91 was grey		27	
	3	limestone.		15	
	4			18	
C101	1	porous amber limestone	12	70	0
	2	porous pink limestone	2	149	
	3	porous light grey limestone	14	13	
	4	porous grey limestone	8	38	
	5	dark brown shale	33	292	
	6	smooth tan/grey limestone	11	10	
	7	yellow/tan limestone	14	27	
	8	pink/brown sandstone	6	42	
D01	1	mica	14	593	0.5
	2	green gabbro	10	140	
	3	yellow limestone	19	13	
	4	brown limestone	19	72	
	5	dark brown limestone	10	36	
	6	porous amber/tan limestone	23	27	
	7	tan limestone	5	35	
E11	1	Coarse tan/brown limestone w/quartz	92	215	0.5
	2	fine-grained tan limestone		767	
	4	fine-grained tan limestone w/quartz		415	
	3	yellow/tan limestone w/quartz crystals	8	36	
	5	yellow/tan limestone w/quartz crystals		42	
G01	1	brown limestone	36	58	1.5
	2	dark grey limestone	28	60	
	3,4	tan limestone	36	21,20	

Table A1. Continued

Core No.	Sample No.	Description of Sample	% Present in core	EDF Value	PCA Rating of Pavement
G31	1	granite	24	111	1.0
	2	dark brown shale	3	186	
	3	light brown slate	21	36	
	4	grey quartzite	25	556	
	5	rust sandstone	8	72	
	6	chert	18	574	
	7	black quartzite	1	50	
H01	1	black mica	20	398	0.5
	2	orange limestone	24	15	
	3	dark grey dolomite	18	477	
	4	dark brown limestone	9	69	
	5	quartzes	6	264	
	6	dark brown limestone w/quartz	3	508	
	7	fine-grain tan limestone	11	45	
	8	quartzites	9	401	
H11	1,3	fine-grain tan limestone	84	328,154	0
	2,4	coarse tan limestone	16	87,74	
H51	1,2	limestone	100	150,174	0.5
J21	1	pink granite	7	296	
	2	opaque quartz	3	296	0
	3	rust/grey sandstone	6	95	
	4	chert w/limestone	6	41	
	5	grey slate	15	123	
	7	yellow/brown sandstone	63	61	
K21	1	slate	16	405	1.0
	2,3	quartz	24	111,75	
	4	brown limestone	33	31	
	5	white rock	13	31	
	7	pink limestone	14	28	
L01	1	yellow limestone	6	20	2.0
	2	grey/tan quartzite	8	395	
	3	black quartzite	31	547	
	4	black slate	14	271	
	5	tan limestone	41	83	

Table A1. Continued

Core No.	Sample No.	Description of Sample	% Present in core	EDF Value	PCA Rating of Pavement
L03	1	granite	7	162	3.0
	2	black quartzite	20	438	
	3	yellow limestone	10	12	
	4	dark brown slate	3	203	
	5	tan dolomite	27	28	
	6	chert	20	76	
	7	dark brown limestone	3	313	
	8	chert/limestone mixture	10	11	
M01	1,4	dolomite	40	337,412	1.0
	2	limestone	7	22	
	3	chert	5	33	
	5	brown limestone	28	52	
	6	white limestone	20	103	
M11	1,4	white limestone	12	31,27	3.5
	2,6	dark grey limestone	24	690,562	
	3,5	tan limestone	64	56,69	
M21	1,5	white limestone	18	134,125	2.5
	2,4	brown limestone	54	32,27	
	3,6	dolomite	28	262,236	
M31	1	fine-grain yellow quartz	5	1043	5.0
	2	coarse yellow quartz	8	232	
	3	pink granite	16	96	
	4	brown/white chert	8	58	
	5	rose quartz	8	281	
	6	black limestone	5	441	
	7	tan limestone	32	36	
	8	brown limestone	12	37	
M51	1	granite	20	181	3.5
	2,4	black quartzite	24	540,251	
	3	brown sandstone	20	86	
	5	orange limestone	12	14	
	6	white limestone	20	36	
	7	orange/yellow quartzite	4	133	
N01	1	dark grey quartzite	10	728	4.0
	2	yellow sandstone	3	117	
	3	rust limestone	3	11	
	4	white quartzite	14	318	
	5	dark brown rock	3	43	
	6	tan sandstone	33	21	
	7	orange quartzite	17	120	
	8	granite	17	80	

Table A1. continued

Core No.	Sample No.	Description of Sample	% Present in core	EDF Value	PCA Rating of Pavement
N21	2	red granite	4	124	2.5
	3	white limestone	9	36	
	4	tan limestone	37	96	
	5,7	black sandstone	24	526,276	
	6	dark grey slate	13	630	
	8	yellow limestone	13	24	
N31	1,2	dolomite	94	262,332	0
	3,4	yellow limestone	6	29,27	
N41	1	pink granite, quartz	24	207	0
	2	brown slag-like limestone	10	75	
	3	brown limestone	10	88	
	4	grey sandstone	18	28	
	5	dark grey slate	16	529	
	6	tan/grey quartzite	10	607	
	7	grey limestone	12	28	
N51	1	brown/white chert	13	54	1.0
	2	orange limestone	32	9	
	3	black quartzite	7	597	
	4	pale yellow quartz	7	446	
	5	granite	26	420	
	6	dark brown limestone	15	255	
O41	1,3	rust limestone	13	14,13	0
	2,4	brown limestone	87	486,283	
P01	1	pink granite	16	138	0
	2	yellow sandstone	13	77	
	3	pale yellow limestone	9	26	
	4	tan limestone	16	111	
	5	green/grey limestone	6	496	
	6	grey limestone	31	110	
P11	1,3	purple granite	5	148,65	1.0
	2	yellow limestone	23	20	
	4	grey limestone	25	41	
	5	dark grey sandstone	15	624	
	6	black granite/mica	15	404	
	7	brown chert	17	28	
R01	1	amber limestone	44	21	0.5
	2	dark brown sandstone	6	739	
	3	green/grey sandstone	13	121	
	4	coarse tan/yellow limestone	25	28	
	6	granites, quartzes	12	207	

Table Al. continued

Core No.	Sample No.	Description of Sample	% Present in core	EDF Value	PCA Rating of Pavement
R11	2	grey limestone	23	163	1.0
	3	brown/orange sandstone	36	32	
	4	black quartzite	18	245	
	5	black slate	5	535	
	6	grey/brown limestone	18	118	
R31	1,6	amber limestone	41	47,59	1.0
	2,4	tan limestone	18	130,114	
	3,5	white limestone	41	64,48	
S01	1	quartz	2	552	1.5
	2	mica	8	96	
	3	grey quartzite	12	465	
	4	pink granite	8	163	
	5	black slate	8	507	
	6	rust limestone/chert/quartz mixture	7	23	
	7	tan limestone	25	21	
	8	brown limestone	30	58	
S11	1	pink granite	19	147	1.0
	2	pink sandstone	16	36	
	3	white crystalline limestone	6	25	
	4	tan limestone	31	16	
	5	yellow/green gabbro	6	27	
	6	black slate	13	441	
	7	purple quartzite	3	264	
	8	yellow chert	6	8	
T03	1,4	grey limestone	61	55,49	2.5
	2,3	yellow sandstone	39	29,30	
T11	1	pink granite	7	170	1.5
	2	coarse amber limestone	13	40	
	3	black quartzite	3	892	
	4	black slate	10	689	
	5	off-white limestone	20	21	
	6	tan limestone	40	34	
	7	tan/white chert	7	375	
T21	1	granite	12	103	1.0
	2	white limestone	17	12	
	3	grey limestone	22	68	
	4	dark grey slate	9	503	
	5	white sandstone	7	46	
	6	tan limestone	33	15	

Table A1. continued

Core No.	Sample No.	Description of Sample	% Present in core	EDF Value	PCA Rating of Pavement
T41	1	pink granite	13	134	1.5
	2	porous brown limestone	13	30	
	3	tan limestone	36	21	
	4	black quartzite	13	318	
	5	amber/tan limestone	15	31	
	6	grey limestone	10	43	
T43	1	dark red quartz	17	166	1.5
	2	white limestone	12	29	
	3	pale yellow limestone	11	19	
	4	dolomite with quartz	9	56	
	6	dark grey limestone	9	271	
	7	tan limestone	42	18	
T51	1	pale yellow limestone	12	19	2.5
	2	tan limestone	53	21	
	3	dark grey dolomite	9	325	
	4	brown limestone	10	46	
	5	grey chert	16	424	
T53	1	dark quartzite	17	288	2.0
	2	tan limestone	44	27	
	3	white limestone	9	25	
	4	light grey limestone	7	51	
	5	crystalline grey limestone	13	100	
	6	rust limestone	10	22	

TABLE A2. Descriptions of Slag Samples and PCA Samples

Core No. (Sample No.)	Test No.	Description of Sample	EDF Rating	PCA Rating of Pavement
A91	1	brown slag	35	0.5
	2	nonporous khaki slag	288	
	3	tan slag	13	
	4	olive slag	78	
	5	nonporous blue/grey slag	42	
	6	brown/black quartzite	125	
A91	A1	olive slag	28	0.5
	A2	olive slag	7	
	A3	olive slag	6	
	A4	olive slag	7	
	B1	tan slag	79	0.5
	B2	tan slag	11	
	B3	tan slag	20	
	B4	tan slag	11	
	C1	khaki slag	246	0.5
	C2	khaki slag	253	
	C3	khaki slag	201	
B11	1	brown slag	13	0
	2	nonporous green slag	141	
	3	grey slag	47.	
	4	brown slag	31	
B11	A1	blue/grey slag	22	0
	A2	blue/grey slag	87	
	A3	blue/grey slag	32	
	A4	blue/grey slag	62	
	B1	grey/black slag	19	0
	B2	grey/black slag	153	
	B3	grey/black slag	25	
	B4	grey/black slag	23	
B21	1	black slag	53	0.5
	2	brown/grey slag	33	
	3	fine-pored slag	29	
PCA-A	1	limestone	48	0
	2	limestone	50	
PCA-B	1	limestone	33	1.0
	2	limestone	30	

Table A2. continued

Core No. (Sample No.)	Test No.	Description of Sample	EDF Rating	PCA Rating of Pavement
PCA-C	1	dolomite	29	1.0
	2	dolomite	29	
PCA-D	1	dolomite	33	1.11
	2	dolomite	31	
PCA-E	1	limestone	60	1.02
	2	limestone	52	

APPENDIX B

This appendix contains general information on all of the samples tested. This information includes the contract, if applicable, the location of the pavement or sample, the age of the pavement or sample, the type of coarse aggregate tested, and the maximum size of the aggregate.

The following is a list of tables contained herein:

Table B1. General Information on Highway Samples

Core No. (Sample No.)	Contract No.	Location	Age When Cored (Years)	Agg. Type	Max. Agg. Size (inches)
A001	R-10572	SR 25 north of Lafayette, IN; eastbound lane; 500 feet east of I65	3	gravel	2½
A11	R-7633	I65 north of Lafayette, IN: northbound lane; milemarker 184	12	stone	1½
A81	R-6600	I65 north of Lafayette, IN; northbound lane; milemarker 245	15	stone	2½
A91	R-6414	I65 north of Lafayette, IN; northbound lane; milemarker 247	16	slag	2½
B01	R-7525	I94; eastbound lane; milemarker 21	13	stone	1½
B11	R-7883	I94; eastbound lane; milemarker 31	10	slag	1½
B21	R-8476	I94; eastbound lane; milemarker 36	9	slag	1½
C01	R-5180	US 24 just east of intersection of US 24 & SR 13; westbound lane	20	stone	2½
C11	B-5213	I69; southbound lane; milemarker 130	19	gravel	2½

Table B1. Continued

Core No. (Sample No.)	Contract No.	Location	Age When Cored (Years)	Agg. Type	Max. Agg. Size (inches)
C31	R-6267	I69; southbound lane; milemarker 90	16	stone	2½
C41	R-6183	I69; southbound lane; milemarker 73	15	stone	1½
C53	R-5859	SR 18 just beyond interchange I69-SR 18; eastbound lane	17	stone	2½
C61	R-5918	I69; southbound lane; milemarker 41	17	gravel	2½
C81	R-5843	I69; southbound lane; milemarker 30	16	stone	2½
C91	R-5805	I69; southbound lane; milemarker 20	17	stone	2½
C101	R-7274	I69; southbound lane; milemarker 12	13	stone & gravel	2½
D01	R-8221	I65 south of Indianapolis, IN; southbound lane; milemarker 85	10	gravel	1½
DOR	--	Outer wall of a church in Dorchester, England	?	stone	--
E11	R-9219	I64 near Louisville; westbound lane; milemarker 110	9	stone	1½
G01	R-6064	I74 west of Indianapolis, IN; westbound lane; milemarker 50	17	stone	2½

Table B1. Continued

Core No. (Sample No.)	Contract No.	Location	Age When Cored (Years)	Agg. Type	Max. Agg. Size (inches)
G31	R-6269	I74 west of Indianapolis, IN; westbound lane; milemarker 21	17	stone & gravel	2½ 1½
H01	R-7389	I70 west of Indianapolis, IN; eastbound lane; milemarker 12	12	gravel	1½
H11	R-7390	I70 west of Indianapolis,. IN; eastbound lane; milemarker 17	12	stone	1½
H51	R-6931	I70 west of Indianapolis, IN; eastbound lane; milemarker 45	14	stone	2½
J21	R-6852	I70 east of Indianapolis, IN; eastbound lane; milemarker 118	15	gravel	2½
K21	R-5434	I74 east of Indianapolis, IN; westbound lane; milemarker 118	19	gravel	2½
L01, L03	R-2226	US 52 one mile north of SR 47, north of Indiana-polis, IN; north-bound lane	39	gravel	2½
M01	R-6878	US 52 1½ miles south of US 24, near Kentland, IN; southbound lane	14	stone	2½
M11	R-6959	US 52 10 miles south of US 24; northbound lane	14	stone	2½

Table B1. Continued

Core No. (Sample No.)	Contract No.	Location	Age When Cored (Years)	Agg. Type	Max. Agg. Size (inches)
M21	R-5044	US 52 14 miles south of US 24; southbound lane	20	stone	$2\frac{1}{2}$
M31	R-3786	US 52 23 miles south of US 24; southbound lane	26	gravel	$2\frac{1}{2}$
M51	R-3408	US 52 31 miles south of US 24; southbound lane	29	gravel	$2\frac{1}{2}$
N01	R-3785	US 41 south of Terre Haute, IN; just south of SR 246; southbound lane	26	gravel	$2\frac{1}{2}$
N21	R-8978	US 41 south of Terre Haute, IN; 22 miles south of SR 246; southbound lane	8	gravel	$1\frac{1}{2}$
N31	R-5996	US 41 south of Terre Haute, IN; 41 miles south of SR 246; southbound lane	17	stone	$2\frac{1}{2}$
N41	R-3894	SR 65 in Princeton, IN; just beyond CR 200 N; northbound lane	25	gravel	$2\frac{1}{2}$
N51	R-6604	US 41 bypass at Vincennes, IN; northbound lane	14	gravel	$2\frac{1}{2}$
O41	R-7421	I64 near Evansville, IN; westbound lane; milemarker 1	12	stone	$1\frac{1}{2}$

Table B1. Continued

Core No. (Sample No.)	Contract No.	Location	Age When Cored (Years)	Agg. Type	Max. Agg. Size (inches)
P01	R-6960	SR 67 south of Indianapolis, IN; 8 miles south of SR 144; northbound lane	14	gravel	$2\frac{1}{2}$
P11	R-4357	SR 67 south of Indianapolis, IN; $\frac{1}{4}$ mile south of SR 144; northbound lane	22	gravel	$2\frac{1}{2}$
R01	R-8831	SR 37 south of Indianapolis, IN; just south of SR 144; southbound lane	8	gravel	$1\frac{1}{2}$
R11	R-8797	SR 37 south of Indianapolis, IN; 10 miles south of SR 144; southbound lane	8	stone	$2\frac{1}{2}$
R31	R-8369	SR 37 south of Indianapolis, IN; 66 miles south of SR 144; southbound lane	11	stone	$2\frac{1}{2}$
S01, S11	R-7474	US 421 north of Madison, IN; 3 miles north of SR 56; northbound lane	13	gravel	$1\frac{1}{2}$
T03	R-7060	US 50 west of Lawrenceburg, IN; 8 miles east of US 421; eastbound lane	14	stone	$2\frac{1}{2}$
T11	R-7552	US 50 west of Lawrenceburg, IN; 13 miles east of US 421; eastbound lane	12	gravel	$1\frac{1}{2}$
T21,T41, T43,T51, T53	R-4425	US 50 west of Lawrenceburg, IN: 15 to 19 miles east of US 421; eastbound lane	22	stone & gravel	$2\frac{1}{2}$ $1\frac{1}{2}$

Table B2. Portland Cement Association Samples

Core No. (Sample No.)	% of Joints Cracked	Sample Source	Age When Sampled (Years)	Agg. Type	Max. Agg. Size (inches)
PCA-A	0	Portland Cement Association	20	Limestone	$1\frac{1}{2}$
PCA-B	4.8	Portland Cement Association	11	Limestone	$1\frac{1}{2}$
PCA-C	16	Portland Cement Association	14	Dolomite	$1\frac{1}{2}$
PCA-D	77	Portland Cement Association	15	Dolomite	$1\frac{1}{2}$
PCA-E	29	Portland Cement Association	11	Limestone	$1\frac{1}{2}$

APPENDIX C

Appendix C is not included in this copy of the Report. It is as follows:

A copy of the Appendix may be obtained at the cost of duplication by inquiry to:

Joint Highway Research Project
Civil Engineering Building
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West Lafayette, Indiana 47907

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